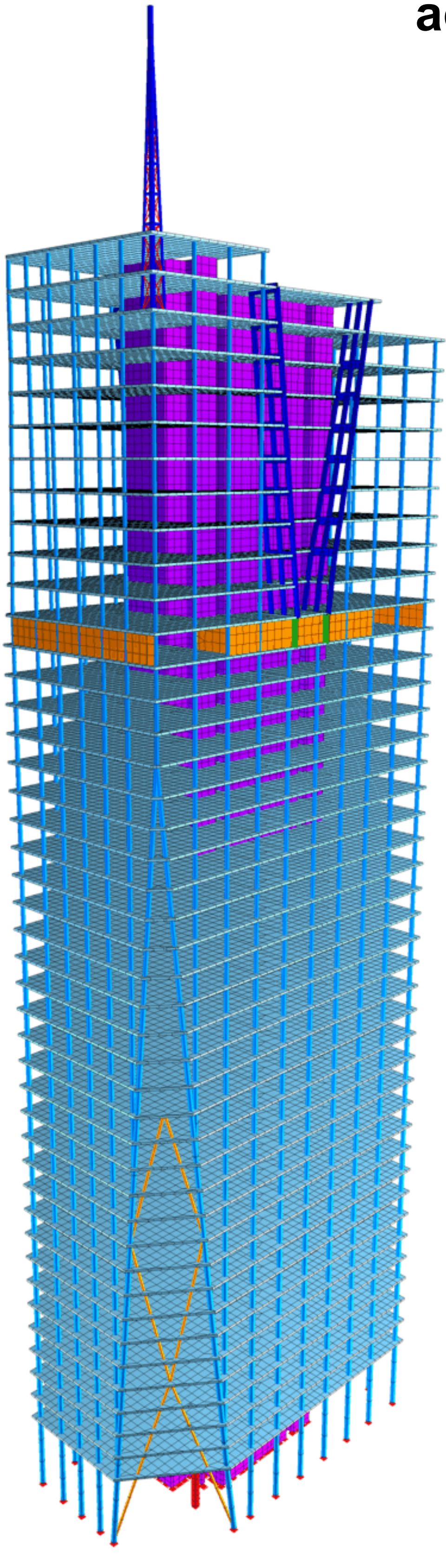
Reinforced and Prestressed Concrete Design

according to EN 1992-1-1

with National Annexes
Austria
Germany
Great Britain
Sweden



InfoGraph
Software für die Tragwerksplanung

Title image: Structure model of the 180 m high 'Europe Tower' in Sofia, Bulgaria.
Courtesy of IDN Ingenieurbüro DOMKE Nachf., Duisburg, Germany.

Contents

EN 1992-1-1 Design

Basics

The reinforced concrete and prestressed concrete design specified in EN 1992-1-1 (Eurocode 2) can be used for buildings and engineering constructions under observance of the following standards:

- EN 1992-1-1:2004/A1:2014 as the base document
- DIN EN 1992-1-1:2015 with the National Annex Germany 2015-12
- OENORM EN 1992-1-1:2015 with the National Annex Austria B 1992-1-1:2018-01
- SS EN 1992-1-1:2014 with the National Annex Sweden 2019-01 (EKS 11)
- BS EN 1992-1-1:2014 with the National Annex Great Britain 2015-07

The desired rule is selected in the *Design Codes* dialog in the *Options* menu. The relevant entry, calculation and results dialogs appear depending on which rule is selected. When selecting the material the following alternatives are available:

- *C12/15-EN-D* to *C100/115-EN-D*, *LC12/13-EN-D* to *LC80/88-EN-D* and the user-defined material *CX-EN-D* for design in accordance with DIN EN 1992-1-1
- *C12/15-EN* to *C90/105-EN*, *LC12/13-EN* to *LC80/88-EN* and the user-defined material *CX-EN* for design in accordance with the other standards

Permitted structure models include beam, area and solid structures. Prestressed structures can only be checked in the FEM module.

Differing components can be combined in a structure model:

- Non-prestressed components
- Prestressed components with subsequent bond
- Prestressed components without bond
- Components with external prestressing
- Mixed-construction components

The design is carried out after the static calculation. To do so, you need to assign the calculated load cases to the actions in accordance with EN 1991:2002 (Eurocode 1), Part 1. The program will take into account the preset safety factors and combination coefficients defined in EN 1990:2021 (Eurocode 0) for the desired design situations to automatically calculate the decisive design internal forces for either the entire system or a group of selected elements.

The actions and check selection dialogs can be opened from the analysis settings. Detailed check specifications and reinforcement data must be entered during section definition.

For beams and design objects, all checks are carried out at the polygon section. In addition, composite sections can be verified in the ultimate limit state. For general notes on using design objects, refer to the relevant chapter in the manual.

In the *EN 1992-1-1 Design* folder of the database and the national variants folders, a single design can also be performed for the user-defined polygon and composite sections.

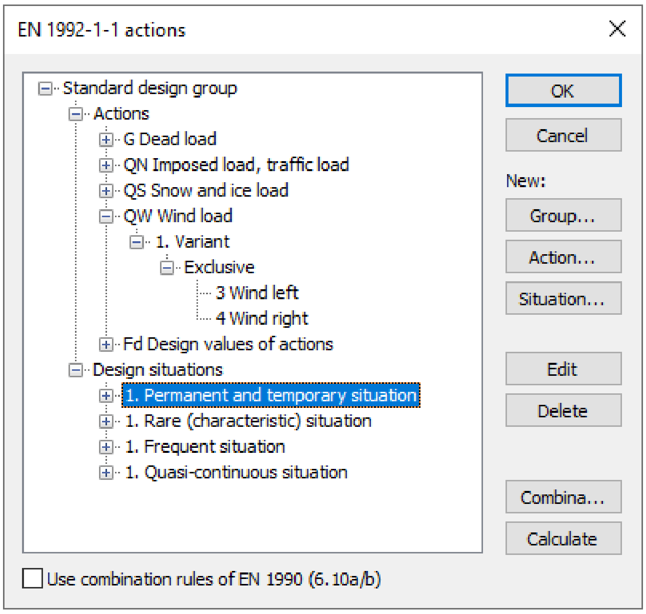
The EN 1992-1-1 guidelines are primarily cited for the following explanations. Reference to the relevant national settings is only made if they contain different or complementary rules. The passages in question are marked by a vertical line left of the text.

Input

Actions and Design Situations

The design values of the load are calculated based on the internal forces of individual load cases and load case combinations. For this the existing load cases and load case combinations must be assigned to actions. These actions are then used to establish the desired design situations.

The following dialog is opened from the database or the *Settings* in the *Analysis* menu.



Action dialog for EN 1992-1-1 (national variants corresponding)

Action...
Open the dialog for entering new actions:

- Permanent actions (G, GE, GH)
- Prestressing (P)
- Creep and shrinkage, relaxation (CSR1, CSR2). These actions are only available if a P action has been defined. In the combinations they are treated, along with P, as a single action.
- Variable actions (QN, QS, QW, QT, QH, QD)
- Accidental actions (A)
- Actions due to earthquakes (AE)
- Design values of actions (Fd)
  The assigned load cases should contain a design-relevant set of loads with partial safety factors and combination coefficients such as for example a load group to take into account nonlinear effects. The selected load cases are combined exclusively.
- Cyclic fatigue actions (Qfat)

Group...
Open the dialog for entering a new design group. According to e.g. standard EN 1991-1-1, Chapter 6.2.2 (2), certain components (sections) may be designed with reduced imposed loads. Therefore, variable actions (Q) and design situations can be changed here.

Situation...
Open the dialog for entering new design situations. Situations must be classified as either a construction stage or a final state in order to control the checking process. For prestressed concrete structures with subsequent bond, you can specify that the tendons are still ungrouted.

Edit
Open the Edit dialog for the selected action or situation.

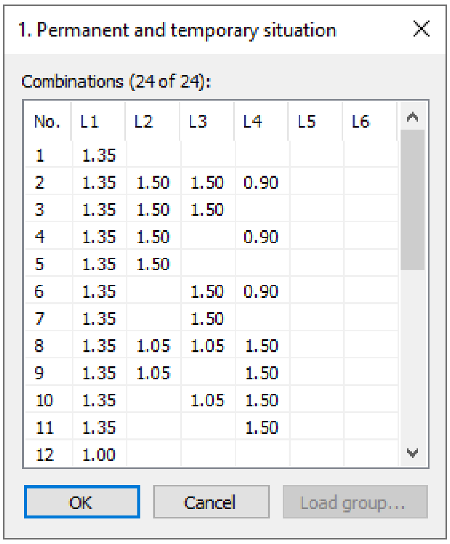
Delete
Delete the selected action or situation.

Combinations...

Opens a dialog that contains the first 999,999 load case variants to be combined for the selected design situation and includes an option to create load groups for selected variants. These variants can be used for second-order theory analysis or nonlinear analysis.

The following example shows the total variants of the *permanent and temporary situation* according to Eq. (6.10) to be examined with the load cases (L1...L6) involved and their weighting factors.

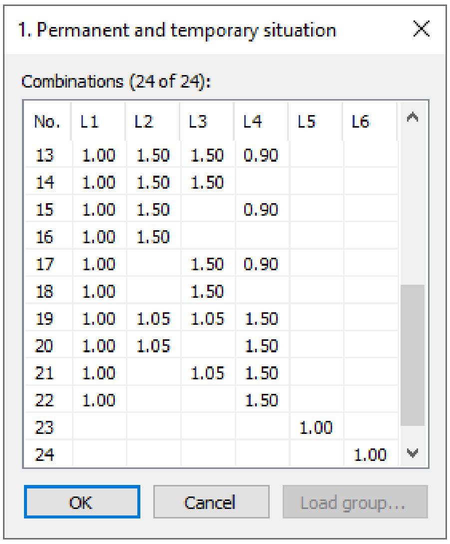
Actions	Load cases	γ_{sup}	γ_{inf}	ψ_0
Dead load	1	1.35	1.0	-
Imposed load, traffic load	2, 3 (inclusive)	1.5	0	0.7
Wind load	4	1.5	0	0.6
F_d Design values of actions	5, 6	1.0	1.0	-


1. Permanent and temporary situation

Combinations (24 of 24):

No.	L1	L2	L3	L4	L5	L6
1	1.35					
2	1.35	1.50	1.50	0.90		
3	1.35	1.50	1.50			
4	1.35	1.50		0.90		
5	1.35	1.50				
6	1.35		1.50	0.90		
7	1.35		1.50			
8	1.35	1.05	1.05	1.50		
9	1.35	1.05		1.50		
10	1.35		1.05	1.50		
11	1.35			1.50		
12	1.00					

OK Cancel Load group...


1. Permanent and temporary situation

Combinations (24 of 24):

No.	L1	L2	L3	L4	L5	L6
13	1.00	1.50	1.50	0.90		
14	1.00	1.50	1.50			
15	1.00	1.50		0.90		
16	1.00	1.50				
17	1.00		1.50	0.90		
18	1.00		1.50			
19	1.00	1.05	1.05	1.50		
20	1.00	1.05		1.50		
21	1.00		1.05	1.50		
22	1.00			1.50		
23					1.00	
24						1.00

OK Cancel Load group...

Calculate

Calculate the defined design situations. Once calculated, the extremal results (internal forces, support reactions) can be accessed for all situations in the database. This allows you to evaluate the results without having to execute the checking module. Each time you execute the checking module, all results will be automatically recalculated using the currently valid actions and then stored in the database for the elements to be checked.

Use combination rules of EN 1990 (6.10a/b)

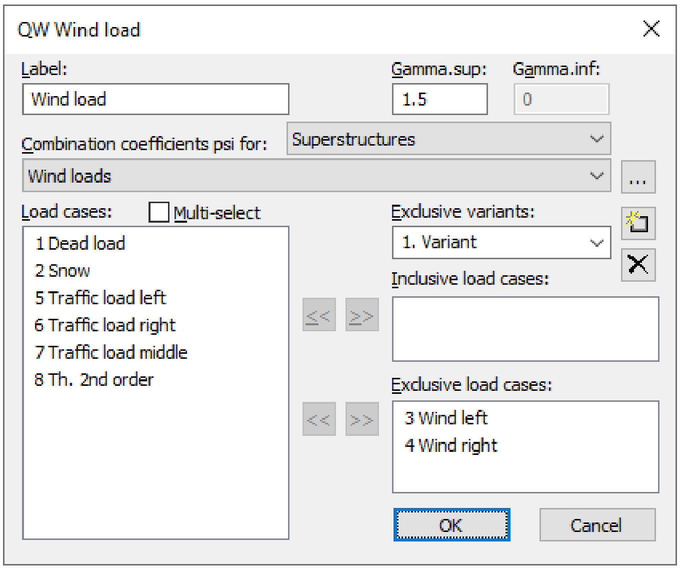
Optionally the Eq. (6.10a/b) are used for the combination of the permanent and temporary situation, otherwise Eq. (6.10).

The following table demonstrates how the situations are used in the various checks. The numbers refer to the Chapters of the EN 1992-1-1 standard.

Situation	Ultimate limit state	Chapter	Serviceability limit state	Chapter
Perm. and temp. Accidental Earthquake	Longitudinal reinf. Lateral reinf. Torsional reinf.	6.1 6.2 6.3		
Characteristic (rare)	Robustness reinf. (EN 1992-2, 6.1 (110))	9.2.1.1	Concrete compr. stress Reinforcing steel stress Prestressing steel stress Crack width, prestr. with immed. bond	7.2 (2) 7.2 (5) 7.2 (5) 7.3.1DE
Frequent	Fatigue, simplified	6.8.6 (2)	Decompr. Class XD1-XS3 Crack width, prestr. with bond	7.3.1 7.3.1
Quasi-continuous			Concrete compr. stress Prestressing steel stress Decompr. Class XC2-XC4 Crack w., reinf.concr. & prestr. w/o b. Deformations	7.2 (2) 7.2 (5)DE 7.3.1 7.3.1 7.4
Fatigue	Fatigue reinf. steel Fatigue prestr. steel Fatigue concrete	6.8.4 6.8.4 6.8.7 (1)		

Definition of an Action

The illustration below shows an example of the dialog field for entering a variable action. The dialog fields for other action types are of a similar appearance.



Name

User-defined label for the action.

Gamma.sup, Gamma.inf

Partial safety factors γ_{sup} and γ_{inf}. The nationally valid values are suggested based on EN 1990, Table A.1.2 (B). For action P, the country-specific coefficients according to EN 1992-1-1, Chapter 2.4.2.2 (1), and for the other actions the nationally valid values according to EN 1990, Table A.1.2 (B), are proposed.

DIN EN 1992-1-1:

In accordance with 2.3.1.3 (4) a partial safety factor for settlements $\gamma_{G,Set} = 1.0$ can be assumed for the linear-elastic determination of internal forces with stiffnesses of uncracked sections.

SS EN 1990:

The program suggests the partial safety factors as they result in accordance with Section A, Article 11, for safety class 3 from $\gamma_d \cdot \gamma_{sup}$ with the reduction factor $\gamma_d = 1.0$ as per Article 14. If required, lower safety classes can be taken into account entering lower values.

Combination coefficients psi for:

Input fields for selecting the combination coefficients for variable actions according to EN 1990. The default number values are based on the national specifications in Table A.1.1 of the standard. Click the ... button to view and edit the selected combination coefficients ψ_0, ψ_1 and ψ_2.

Load cases

List of the possible load cases or load case combinations. Select items by highlighting them and clicking the >> button or use drag & drop.

Multi-select

Load cases and combinations can be added to the actions more than once.

Exclusive variants

Variable actions may consist of multiple exclusive variants that are mutually exclusive. The variants themselves contain both inclusive and exclusive parts. You can add or delete action variants with the or buttons.

Inclusive load cases

Selected load cases and combinations that can have a simultaneous effect.

Exclusive load cases
Selected load cases and combinations that are mutually exclusive.

Prestressing loss from relaxation of prestressing steel
The prestressing loss is defined as a constant percentage reduction of prestress.

CS as constant reduction of prestress
As an alternative to defining CS load cases, you can allow for the effect of creep and shrinkage by defining a constant percentage reduction of prestress.

Internal prestressing
Selected load cases that describe internal prestressing. The reactions of the individual load cases are added together.

External prestressing
Selected load cases that describe external prestressing. The reactions of the individual load cases are added together.

Partial Safety Factors

The partial safety factors of the construction materials are preset with the nationally applicable values as specified in EN 1992-1-1, Table 2.1. In the design situations due to earthquakes, the safety factors of the accidental design situation may be assumed in accordance with EN 1998-1, Chapter 5.2.4 (3), if the strength loss is taken into account when determining the material properties. Otherwise, the factors of the permanent and temporary design situation must be applied in accordance with Chapter 5.2.4 (2).
The partial safety factors for the actions are specified in the definition of the actions based on EN 1990, Table A.1.2(B).

OENORM B 1998-1:

In design situations resulting from earthquakes, the factors for construction materials according to OENORM B 1998-1, Chapter 5.2.4 (3), apply.

DIN EN 1998-1:

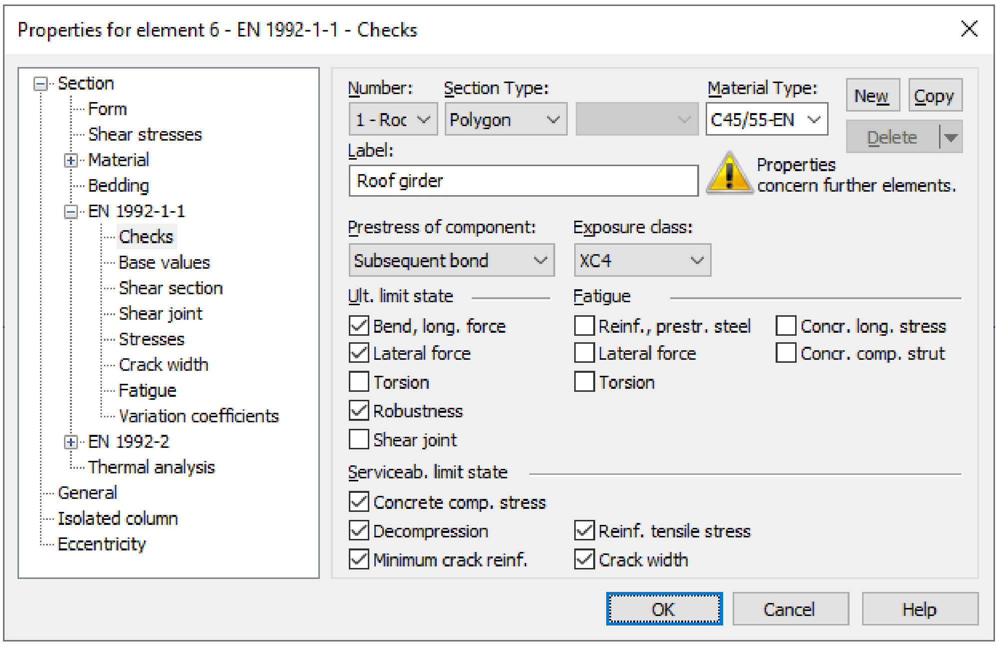
In the design situations due to earthquakes, according to the NDP to 5.2.4 (1) and (3), the safety factors of the permanent and temporary design situation generally apply.

Section Inputs

The section inputs contain all of the specific settings made for checks in the ultimate limit and serviceability states. In addition to these specifications, the selected material properties and the properties of the reinforcing steel are also relevant for the design. An overview of the design specifications can be accessed in the *EN 1992-1-1 Design* folder of the database and in the folders of the national variants.

Checks

The following dialog is used to define which checks are available for the cross-section in the ultimate, fatigue and serviceability limit states. For composite sections, the selection is limited to the load-bearing capacity checks. The analysis settings allow to override this selection for the entire structure.



Check selection for EN 1992-1-1 (national variants corresponding)

Prestressing of the component

The type of prestressing can be selected for each section separately:

- *Not prestressed*
- *Subsequent bond*
- *Without bond*
- *External*
- *Mixed construction*

Exposure class

The check conditions for the decompression and crack width check are grouped by exposure class in EN 1992-1-1, Chapter 7.3, Table 7.1N. A component can be assigned to an exposure class based on the information provided in Table 4.1 of the standard.

SS EN 1992-1-1:

In addition, the service life class as per Article 10 can be selected to determine the crack width according to Table D-2 and the crack safety factor according to Table D-3.

Robustness

This check determines the minimum reinforcement against failure without notice (robustness reinforcement) based on EN 1992-1-1, Chapter 5.10.1 (5)P with the method specified for prestressed concrete bridges in EN 1992-2, Chapter 6.1 (109), Equation (6.101a). It thus offers an alternative to minimum reinforcement as per EN 1992-1-1, Chapter 9.2.1.1 (1), Equation (9.1N). The latter can be taken into account when necessary by specifying a base reinforcement in the reinforcing steel description.

DIN EN 1992-1-1:

According to Chapter 9.2.1.1 (1), the ductile component behavior must always be ensured for components with or without prestressing by applying robustness reinforcement.

Steel tensile stresses

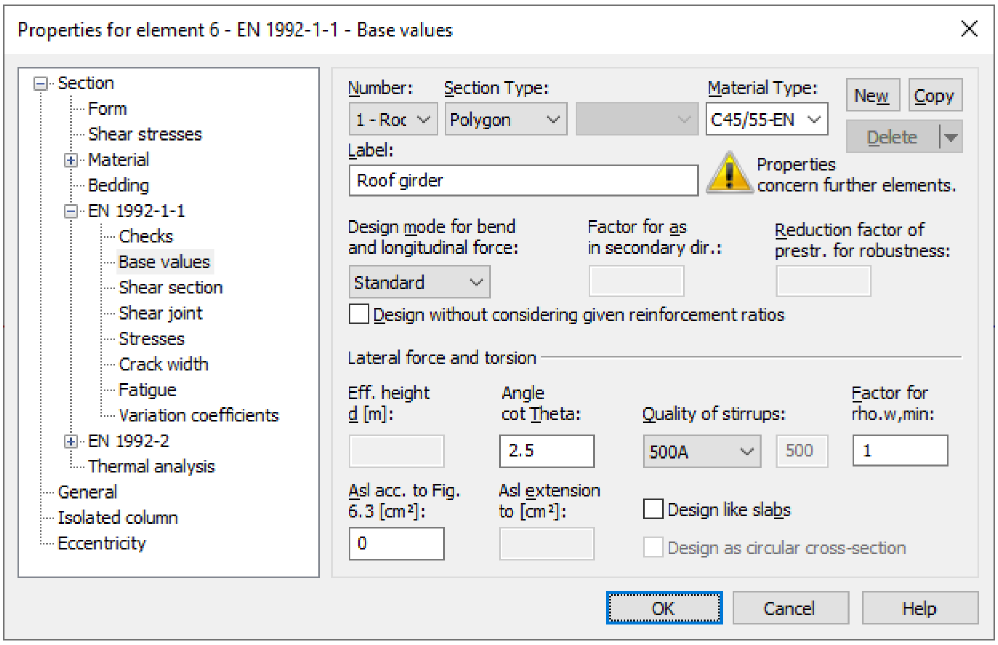
For components with internal prestressing, both the prestressing steel stresses an the stresses of the longitudinal reinforcement are checked.

Minimum crack reinforcement, crack width

The crack width check is carried out according to Chapter 7.3.4. In this check the final longitudinal reinforcement is set as the maximum value from the bending reinforcement, robustness reinforcement and minimum crack reinforcement as per 7.3.2. The latter will be increased automatically if necessary to maintain the crack width.

Base Values

Unless otherwise specified, the base values apply for all checks in the ultimate, fatigue and serviceability limit states.



Design mode

- *Standard*: Standard design mode for bending with normal force throughout the load area. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary.
- *Compression member*: For compression members, a symmetrical design is carried out taking into account the minimum reinforcement according to Section 9.5.2 (2).

Factor for as in secondary direction

According to EN 1992-1-1, Section 9.3.1.1 (2), secondary longitudinal reinforcement of one-way slabs should not be less than 20% of the principal reinforcement. The examination is carried out on the program side with the results of the bending design separately for the upper and lower side of the cross-section. The direction with the largest amount of reinforcement per cross-sectional side defines each principal reinforcement direction. The assignment of the factorized reinforcement in secondary direction then takes place via corresponding reinforcement layers.
DIN EN 1992-1-1:
In the case of two-way slabs, the less stressed direction should be reinforced with at least 20% of the higher stressed direction.

Reduction factor of prestr. for robustness

In the program the regulations of the EN 1992-2, Chapter 6.1 (110) are decisive for the arrangement of the robustness reinforcement. Thus for the determination of the tensile zone the statically determined effect of prestressing is not taken into account. Because this cannot be determined for area elements alternatively the prestress can be reduced by a reduction factor. The specification of an appropriate value is subject to the discretion of the user.

Design without considering given reinforcement ratios

If selected, the reinforcement increase required in the design is performed without taking into account the reinforcement ratios specified by the basic reinforcement.

Effective height

Effective static height for the shear design of area elements [m].

Angle cot Theta

$\cot\Theta$ defines the concrete strut angle according to Chapter 6.2.3 (2), Equation (6.7N). The program will suggest a value of 1 (45° strut angle). You can choose to ignore the suggestion and pick any value within the permissible national limits. Entering a higher number will normally result in a lower necessary lateral force reinforcement A_{sw}, a lower absorbable lateral force $V_{Rd,max}$ and a larger displacement a_1 according to Chapter 9.2.1.3, Equation (9.2).

DIN EN 1992-1-1:
Three calculation methods can be chosen for the check:

- *Standard*: The input value is limited to the range permitted in accordance with Eq. (6.7aDE) for lateral force, torsion and combined loads (method with load-dependent strut angle).
- *Constant*: The check is carried out using the chosen value for $\cot\Theta$ without further limitations (cf. interpretation No. 24 of NABau for DIN 1045-1).
- *Std./45°*: For lateral force $\cot\Theta$ is limited according to Eq. (6.7aDE), for torsion a constant strut angle of 45° is assumed for simplification according to Chapter 6.3.2 (2).

The actual effective angle of the concrete struts is logged for each check location.

OENORM B 1992-1-1:
The concrete strut angle is defined by $\tan\Theta$ and should be limited according to equations (3AT) and (4AT).

SS EN 1992-1-1:
According to Article 15 and differing from Equation (6.7N), for prestressed components the condition $1.0 \le \cot\Theta \le 3.0$ applies.

Asl acc. to Fig. 6.3

The bending tensile reinforcement to be taken into account according to Chapter 6.2.2, Figure 6.3 [cm²].

Asl extension to

You can optionally specify a maximum value for areas and the program will automatically increase the above input value until that maximum value is reached in order to avoid stirrup reinforcement [cm²].

Quality of the stirrups

- *420S*: Reinforcing rod with f_{yk} = 420 MN/m².
- *500A*: Reinforcing rod with f_{yk} = 500 MN/m².
- *500M*: Reinforcing meshes with f_{yk} = 500 MN/m².
- *General information*: Freely definable steel quality [MN/m²].

Design like slabs

Beams or design objects are treated like slabs, which means that a minimum lateral force reinforcement will not be determined as per Chapter 6.2.1 (4), if no lateral force reinforcement is required for computation.

Factor for rho.w,min

The minimum reinforcement level $\rho_{w,min}$ is defined using a factor related to the standard value for beams according to EN 1992-1-1, Chapter 9.2.2 (5).

DIN EN 1992-1-1, OENORM B 1992-1-1:
For slabs with $V_{Ed} > V_{Rd,c}$ at least the 0.6-fold value of the minimum shear reinforcement of beams is necessary.

DIN EN 1992-1-1:
For structured sections with prestressed tension chord the 1.6-fold value is to be applied according to Equation (9.5bDE).

SS EN 1992-1-1:
If the fire safety class is 1 or 2 and no shear reinforcement is required, $\rho_{w,min}$ can be set to zero as per Article 26.

Design as circular cross-section

For circular and annular cross-sections, the lateral force design according to Bender et al. (2010) can be selected as an alternative for the resulting shear force $Q_r = \sqrt{(Q_y^2 + Q_z^2)}$. The corresponding inputs are made on the *Shear Section* dialog page.

Laying measure cv,l

DIN EN 1992-1-1:

In Chapter 6.2.3 (1) the inner lever arm z is limited to the maximum value derived from $z = d - c_{v,l} - 30\,\text{mm}$ and $z = d - 2c_{v,l}$. Note that $c_{v,l}$ is the laying measure of the longitudinal reinforcement in the concrete compressive zone. For $c_{v,l}$ the program will suggest the smallest axis distance of the longitudinal reinforcement to the section edge d_1.

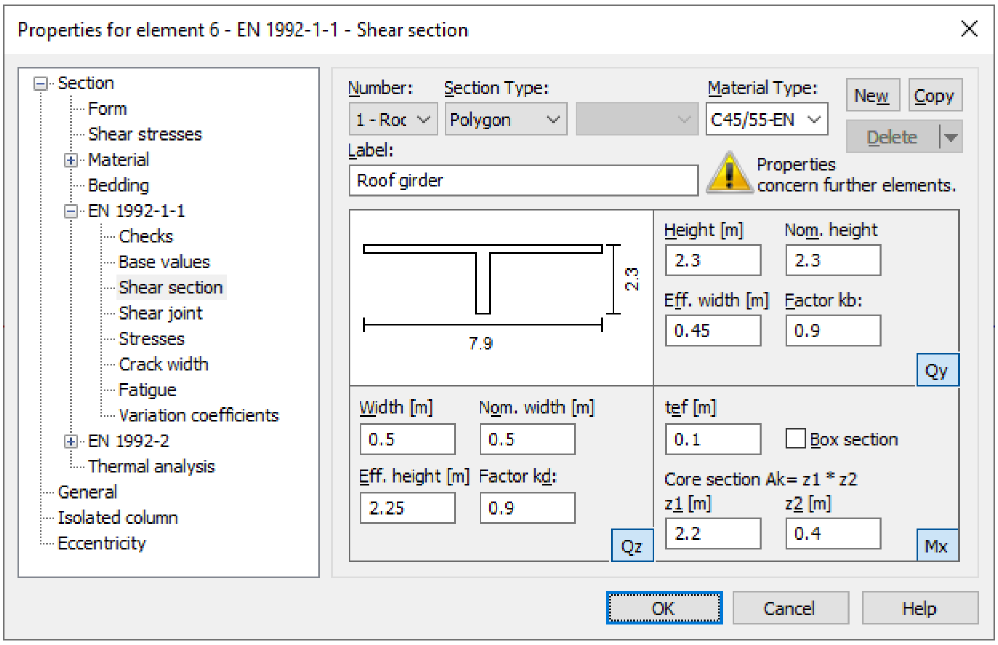
Separate check for x and y direction

DIN EN 1992-1-1:

For two-axes stressed slabs, the lateral force check can be performed separately in the x and y stress directions as described in Chapter 6.2.1 (10). The user is responsible for properly aligning the reinforcement directions.

Shear Section

For polygon and composite sections, additional section dimensions are required for the lateral force and torsion design. These are explained in the following. In case of sections with internal prestressing or with a shape that differs from a rectangle, the dimensions suggested by the program should be reviewed.



Width

Section width for calculating the lateral force load-bearing capacity for Q_z [m].

Height

Section height for calculating the lateral force load-bearing capacity for Q_y [m].

Effective height

Effective static height for calculating the lateral force load-bearing capacity for Q_z [m].

Effective width

Effective static width for calculating the lateral force load-bearing capacity for Q_y [m].

Nominal width, nominal height

The nominal width or height of internally prestressed components as per EN 1992-1-1, Chapter 6.2.3 (6), for including the duct diameter in the calculation of the design value of the lateral load-bearing capacity $V_{Rd,max}$.

Factor kb, Factor kd

Factor for calculating the inner lever arm z from the effective width bn or effective height d in the lateral loadbearing capacity check for Q_y or Q_z.

Core section Ak = z1 * z2

Dimensions of the core section for calculating the torsion reinforcement [m].

tef

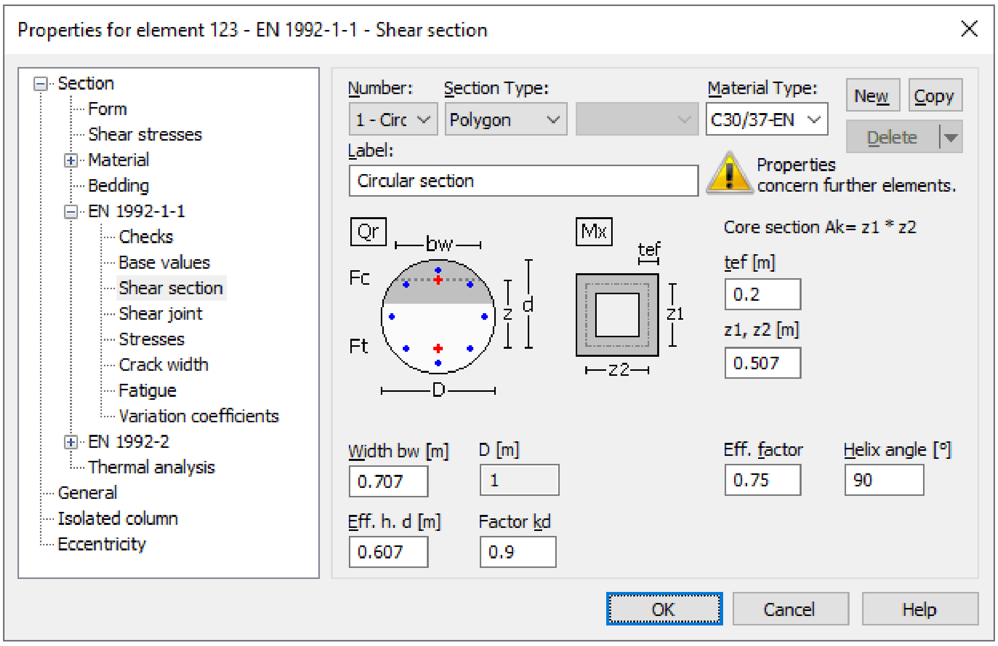
The effective wall thickness of the torsion section according to Figure 6.11 [m].

Box section

Selection of the rules applicable for box sections for the check of the maximum load-bearing capacity according to Chapter 6.3.2 (4) and for the required reinforcement according to Chapter 6.3.2 (5) in case of combined stress from lateral force and torsion.

Circular and annular cross-section

If the circular design according to Bender et al. (2010) was selected for the resulting lateral force Q_r on the *Base values* dialog page, the equivalent cross sections for the shear design must be defined in the following dialog.



Width bw

Effective section width for calculation of the lateral force bearing capacity for $Q_r = \sqrt{(Q_y{}^2 + Q_z{}^2)}$. According to the recommendation of the German Committee for Standardization in Civil Engineering (NABau), the smaller value of the section width at the center of gravity of the steel tensile forces and the concrete compressive forces should be selected for the effective width b_w. For circular cross-sections, the program suggests the dimension of the square inscribed in the circle ($R \cdot \sqrt{2}$) for b_w, and twice the wall thickness for annular cross-sections.

Effective height d

Statically effective height for calculation of the lateral force bearing capacity for Q_r. The program suggests $d = h - d_1$, where the height is set to $h = R \cdot \sqrt{2}$ and d_1 indicates the edge distance of the outer reinforcement layer.

Factor kd

Factor for calculating the inner lever arm z from the effective height d in the verification for Q_r.

Efficacy factor

According to Bender et al. (2010), p. 422, the efficacy factor α_k is stress-dependent ($0.715 \le \alpha_k \le 0.785$) and can be assumed with the mean value $\alpha_k = 0.75$.

Helix inclination

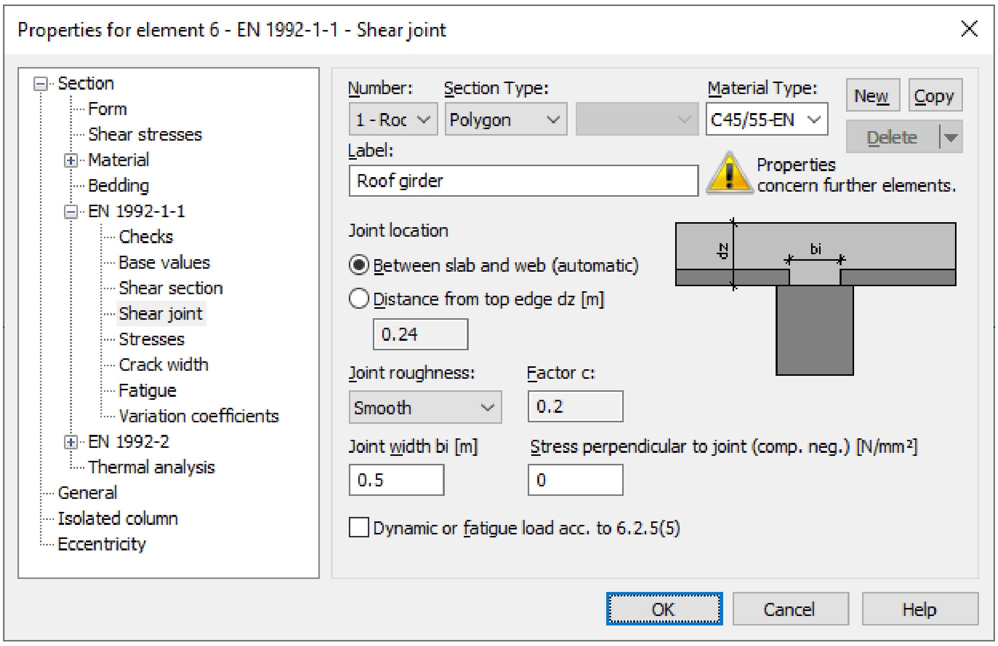
Angle between shear force reinforcement and component axis. When entering an inclination of 90°, annular single stirrups are assumed.

z1, z2, tef

The dimensions z_1, z_2 of the square core cross-section and the effective wall thickness t_{ef} of the torsion box are defined according to EN 1992-1-1, Figure 6.11. The design for torsion is carried out according to the standard for vertical stirrups.

Shear Joint

The shear joint check is available for polygon and composite sections. The input values proposed by the program must be checked by the user and adjusted if necessary.



Joint location

The program can automatically determine the location of the joint at the transition between the slab and the web. Alternatively, the user can define the distance of the joint from the top edge of the cross-section dz [m].

Joint roughness

The roughness of the joint (very smooth, smooth, rough, indented).

Factor c

Factor for determining the shear resistance in the joint, which is specified depending on the joint roughness according to EN 1992-1-1, Chapter 6.2.5 (2) and can only be adjusted by the user if the joint is very smooth.

Joint width bi

Width of the joint over which shear forces are transferred between existing and new concrete [m].

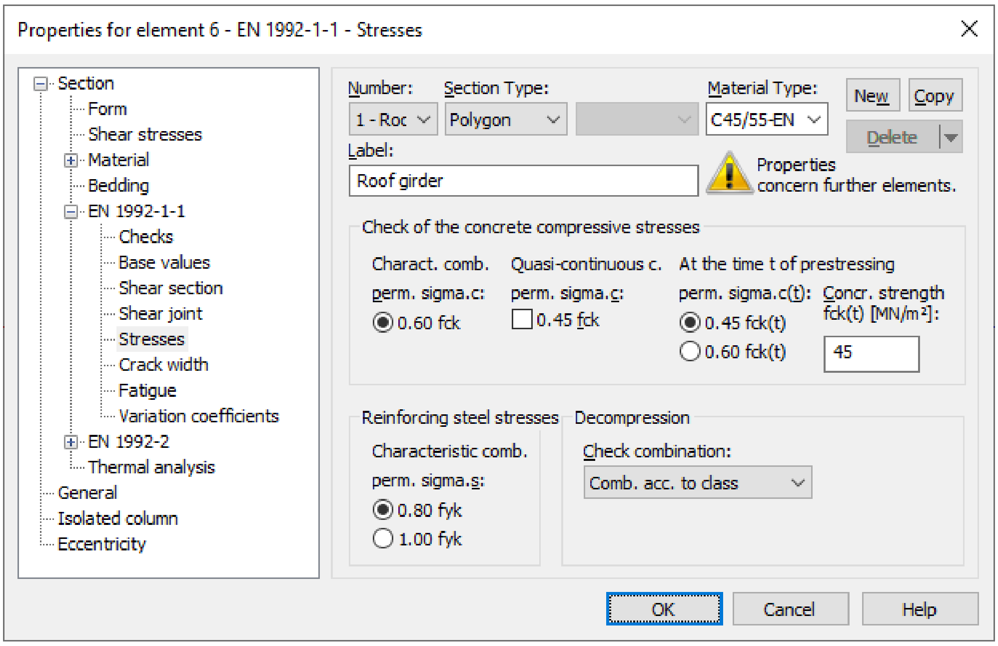
Stress perpendicular to joint (comp. neg.)

Stress σ_n caused by the minimum normal force perpendicular to the joint which can act simultaneously with the lateral force [N/mm²]. Compressive stresses must be entered with a negative sign and are limited in the check according to 6.2.5 (1).

Dynamic or fatigue stress according to 6.2.5(5)

If this option is selected, a dynamic or fatigue stress on the cross-section is assumed and the factor c is adjusted according to 6.2.5 (5).

Stresses



perm. sigma.c

The concrete compressive stress σ_c must be limited to $0.60\, f_{ck}$ under the characteristic action combination in the construction stages and final states according to EN 1992-1-1, Chapter 7.2 (2). If stress in the concrete under quasi-continuous combination does not exceed the limit $0.45{\cdot}f_{ck}$, linear creep can be assumed according to 7.2 (3). If this is not the case, non-linear creep must be taken into account.

perm. sigma.c(t)

Permissible concrete stress $\sigma_{c(t)}$ at time t when prestressing is introduced. If the compressive stress exceeds the value $0.45{\cdot}f_{ck(t)}$, the nonlinearity of the creep should be taken into account according to the standard. The program assumes that prestressing is introduced in design situation 'G+P'.

fck(t)

Concrete compressive strength at time t when prestressing is introduced according to Chapter 5.10.2.2 (5) of the standard [MN/m²].

Reinforcing steel stresses

According to Chapter 7.2 (5) the tensile stresses in the reinforcement may not exceed the value $0.8{\cdot}f_{yk}$ under the characteristic action combination. For stresses resulting from indirect action, the limits can be assumed as $1.0{\cdot}f_{yk}$.

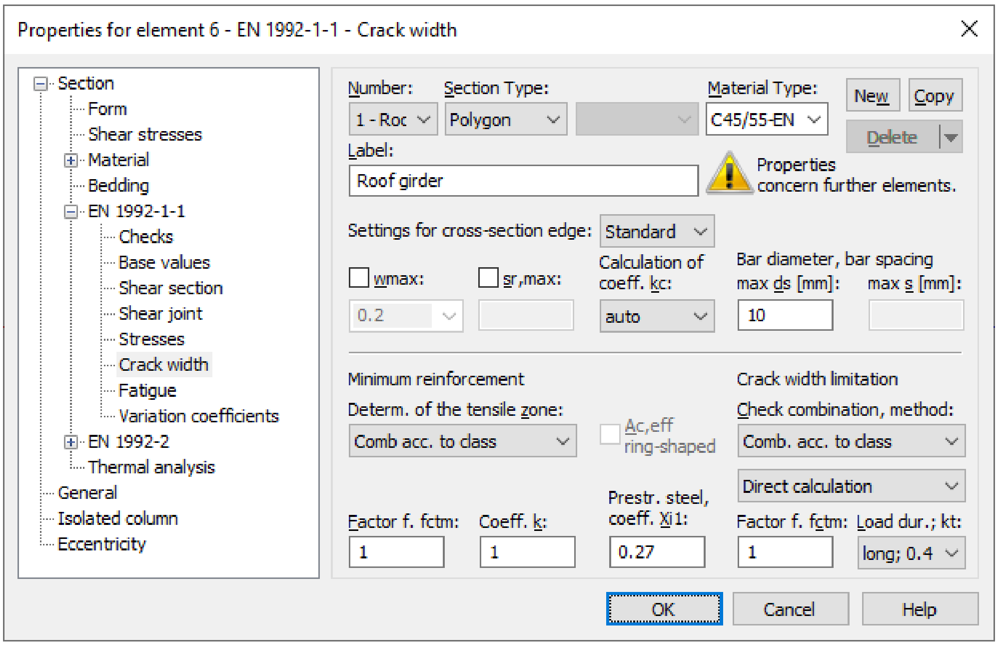
SS EN 1992-1-1:
According to Article 19, the limit $1.0{\cdot}f_{yk}$ can be generally assumed.

Decompression, check combination

The action combination (AC) for the decompression check normally results from the selected exposition class. Alternatively, a deviating combination can be chosen.

Crack Width

These specifications apply to the minimum crack reinforcement calculation and the crack width check.



Section edge

The following properties can be defined differently for the section edges and the reinforcement directions:

w_{max}	limit for the calculated crack [mm].
$s_{r,max}$	largest permissible crack spacing [mm].
k_c	calculation method for coefficient k_c.
max. d_s	largest existing bar diameter [mm].
max. s	largest existing bar spacing [mm].

The following options are available for editing:

Standard	The standard properties are used for the unspecified edges and directions.
Top, bottem, x, y	Definition for the top or bottom edge in the x or y reinforcement direction.
<Add>	Starts the dialog for adding a section edge.
<Delete>	Deletes the displayed section edge.

wmax

Limit for the calculated crack width according to EN 1992-1-1, Chapter 7.3.1, Table 7.1N [mm]. The program will suggest a tabular value according to the national requirements based on the selected exposure class and the prestressing of the component. This value can be modified after the input field is enabled.

SS EN 1992-1-1:

In addition, the service life class is taken into account to determine the suggested value according to Article 20, Table D-2. For prestressed components the tabular values for higher corrosion are taken, for reinforced concrete the values for slight corrosion apply.

sr,max

When calculating the crack width, the crack spacing $s_{r,max}$ is determined by default using Equation (7.11) of the standard. Alternatively, the user can specify an upper limit to take into account any special conditions of Equation (7.14) or Sections (4) and (5) of Chapter 7.3.4, for example.

Coefficient kc

The following methods are available for calculating the coefficient k_c:

auto	For rectangular solid sections, k_c is calculated according to Eq. (7.2), in all other cases according to Eq. (7.3).
web	k_c is calculated according to Eq. (7.2).
chord	k_c is calculated according to Eq. (7.3).

max. ds

Largest existing bar diameter of the reinforcing steel reinforcement for evaluating Equations (7.6N), (7.7N) and (7.11) in Chapter 7.3 of the standard [mm].

max. s

Largest existing bar spacing of the reinforcement for the simplified crack width check as per Chapter 7.3.3 (2) [mm].

Determ. of the tensile zone

You can specify the tensile section where the minimum crack reinforcement as per Chapter 7.3.2 will be placed by selecting either an action combination or a restraint (bending, centrical tension).

Thick component

DIN EN 1992-1-1:

Based on DIN EN 1992-1-1, Chapter 7.3.2 (5), the minimum reinforcement for the crack width limitation in the case of thicker components under centrical restraint can be determined according to Equation (NA 7.5.1). Therewith a reduction compared to the calculation with Equation (7.1) can be achieved.

Minimum reinforcement according to Eq. (16AT)

OENORM B 1992-1-1:

The minimum reinforcement for the crack width limitation under centrical restraint can be determined according to Equation (16AT). Therewith a reduction compared to the calculation with Equation (7.1) can be achieved.

Coefficient k

Coefficient for taking into account nonlinear distributed concrete tensile stresses in the section in Chapter 7.3.2, Equation (7.1). Depending on the flange width or the web height h the value k can be assumed between 0.65 ($h \geq$ 800 mm) and 1.0 ($h \leq$ 300 mm).

DIN EN 1992-1-1:

In case of restraint within the component, k can be multiplied by 0.8 whereby the minimum of the height and the width of the section or section part shall be used for h. For tensile stresses due to restraint generated outside of the component, k = 1.0 applies.

SS EN 1992-1-1:

Depending of the section dimension h (flange thickness resp. web height), the factor k can be assumed between 0.50 ($h \geq$ 680 mm) and 0.90 ($h \leq$ 200 mm) according to Article 4a.

Factor for fctm

This factor is used to specify the effective concrete tensile strength $f_{ct,eff}$ based on the average value of tensile strength f_{ctm}. This is done separately for the minimum reinforcement calculation according to Equation (7.1) and the crack width calculation according to Equation (7.9) of the standard. The tensile strength, which depends on the age of the concrete, is defined in Equation (3.4) of Chapter 3.1.2.

DIN EN 1992-1-1:

If it is not certain wether crack formation will occur within the first 28 days, a tensile strength of at least 3.0 MN/m² for normal concrete and 2.5 MN/m² for lightweight concrete should be assumed for Eq. (7.1). The program meets this requirement if 1.0 is entered for the reduction factor.

Ac,eff ring-shaped

For circular solid and hollow sections, the effective area of the reinforcement $A_{c,eff}$ for the check of the minimum reinforcement and the crack width can be determined ring-shaped according to Wiese et al. (2004).

Coefficient Xi1

The bond coefficient ξ_1 according to Chapter 7.3.2, Equation (7.5), defines the extent to which prestressing steel as per 7.3.2 (3) can be taken into account for the minimum crack reinforcement. It is also used in calculating the effective reinforcement level according to Chapter 7.3.4, Equation (7.10), and thus enters into the direct calculation of the crack width. Data input is blocked for area elements since prestressing steel is normally not taken into account here.

OENORM B 1992-1-1:

The bond coefficient ξ_1 is used to take into account the different bonding behavior of concrete and prestressing steel for the stress checks according to Chapter 7.2 of the standard.

Check combination

The action combination (AC) for the crack width check normally results from the selected exposition class. Alternatively, a deviating combination can be chosen.

Check method

The crack width can be verified either by direct calculation according to Chapter 7.3.4 or simplified by limiting the bar spacing using Table 7.3N. Table 7.3N should only be used for single-layer tensile reinforcement with d_1 = 4 cm under loading (cf. Zilch, Rogge (2002), p. 277; Fingerloos et al. (2012), p. 109; Book 600 of the DAfStb (2012), p. 127). For both methods, a constant average steel strain within $A_{c,eff}$ can optionally be chosen as the basis for calculation.

OENORM B 1992-1-1:

Die The method is applicable to single-layer reinforcement with a bar spacing according to Table 10AT or 11AT. These are valid for concrete covers 25 mm ≤ c_{nom} ≤ 40 mm with bar diameters 8 mm ≤ d_s ≤ 20 mm.

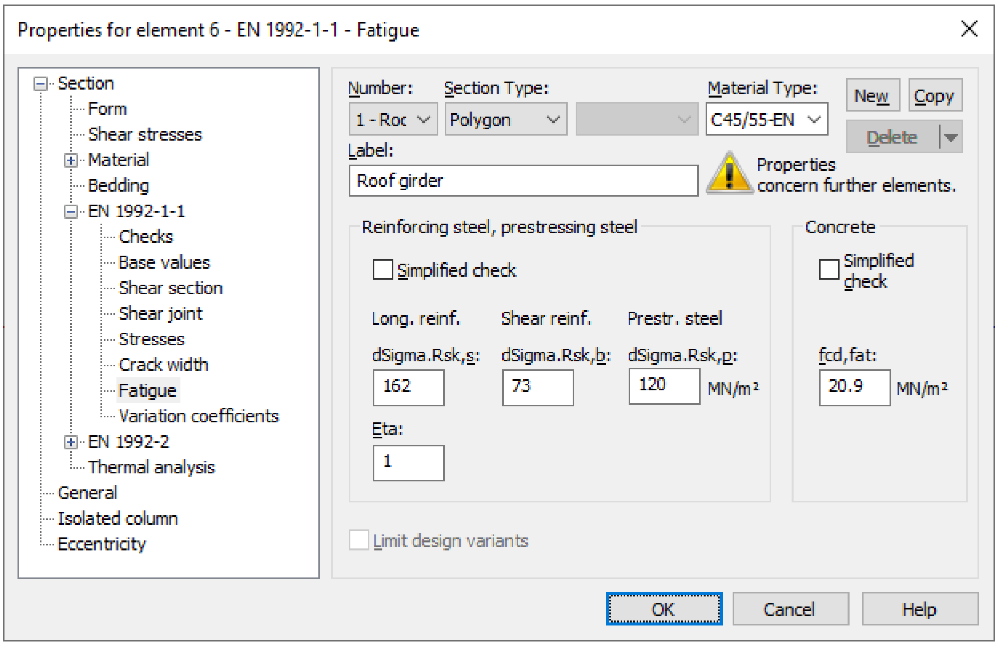
Load duration; kt

This selection defines the factor k_t in Equation (7.9) for crack width calculation.

Note for waterproof concrete structures

For components that are to be designed according to national guidelines for waterproof concrete structures, the permitted crack widths given there can be entered after activating the dialog control w_{max}. If required, the check-relevant action combinations can also be defined differently from the requirements of EN 1992-1-1.

Fatigue



dSigma.Rsk,s, dSigma.Rsk,b

The permissible characteristic stress range $\Delta\sigma_{Rsk}$ (N^*) of the longitudinal reinforcement and shear reinforcement at N^* load cycles according to the S-N curves specified in EN 1992-1-1, Chapter 6.8.4 [MN/m²]. The national decisive value found in Table 6.3N, Row 1 (beam sections) resp. Row 2 (area sections), is suggested in the dialog. For the shear reinforcement, the mandrel diameter is assumed to be four bar diameters.

OENORM B 1992-1-1:

In the dialog, the value according to Table 5, line 1 (beam sections) or line 4 (area sections) is suggested for the longitudinal reinforcement. For the shear reinforcement, the value according to line 1 is suggested.

dSigma.Rsk,p

The permissible characteristic stress range $\Delta\sigma_{Rsk}$ (N^*) of the prestressing steel at N^* load cycles according to the S-N curves specified in Chapter 6.8.4 [MN/m²]. The value found in Table 6.4, Row 4, is suggested in the dialog.

DIN EN 1992-1-1, OENORM B 1992-1-1:

The value for prestressing steel of class 1 is suggested.

Eta

Increase factor η for the reinforcing steel stress of the longitudinal reinforcement. This factor is used to take into account the varying bonding behavior of concrete and prestressing steel as per Chapter 6.8.2 (2)P, Eq. (6.64).

fcd,fat

Concrete compressive strength before onset of cyclic load according to Chapter 6.8.7 (1), Eq. (6.76) [MN/m²]. In general, the following applies:

$$f_{\text{cd,fat}} = k_1 \cdot \beta_{\text{cc}}(t_0) \cdot f_{\text{cd}} \cdot \left(1 - \frac{f_{\text{ck}}}{250}\right) \tag{6.76}$$

with

$$\beta_{\text{cc}}(t_0) = e^{s\,(1-\sqrt{28/t_0})}$$

s Coefficient depending on the cement type.

t_0 Time of the initial stressing of the concrete.

k_1 $= 0.85$

$f_{\text{cd,fat}}$ for $s = 0.2$, $t_0 = 28$ and f_{cd} according to Eq. (3.15) is suggested in the dialog.

DIN EN 1992-1-1, OENORM B 1992-1-1, SS EN 1992-1-1:

k_1 $= 1.0$

BS EN 1992-1-1:

For the proposed value of $f_{\text{cd,fat}}$, f_{cd} is determined with $\alpha_{\text{cc}} = 1.0$ in Eq. (3.15).

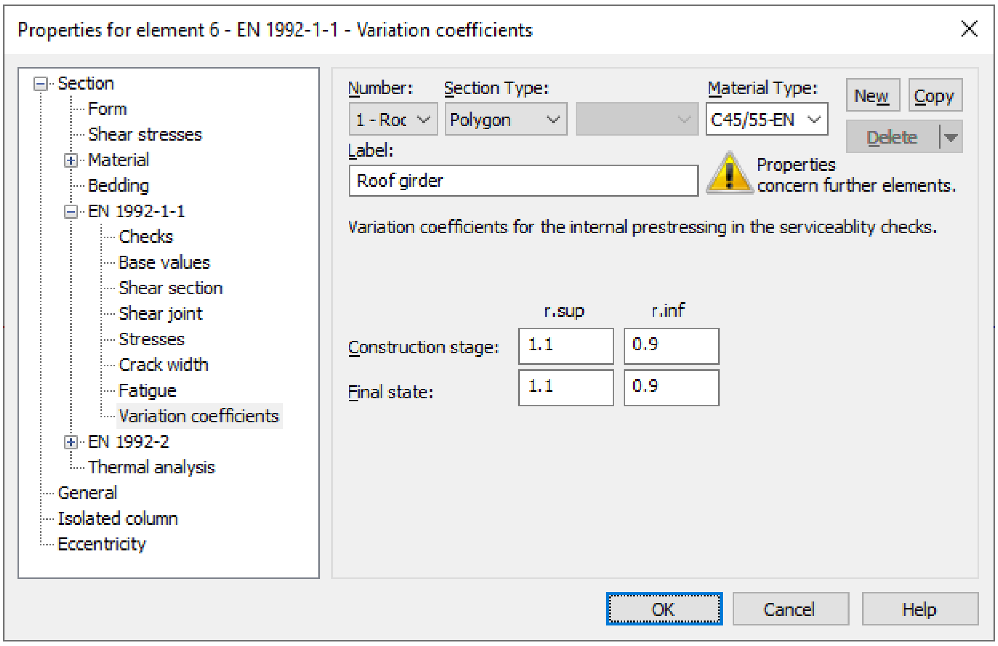
Simplified check

The simplified check according to Chapter 6.8.6 (2) bases on the frequent action combination including the traffic loads at serviceability limit state. The method for concrete is defined in Chapter 6.8.7 (2), the permissible stress ranges for steel are suggested according to Chapter 6.8.6 (1) in the dialog. For shear reinforcement this value is reduced analogous to Table 6.3N.

Limit design variants

For area elements, the variants for determining the stress range can be limited to the corresponding sets of design internal forces. For more information see chapter '*Check Against Fatigue > Special Characteristic of Shell Structures*'.

Variation Coefficients



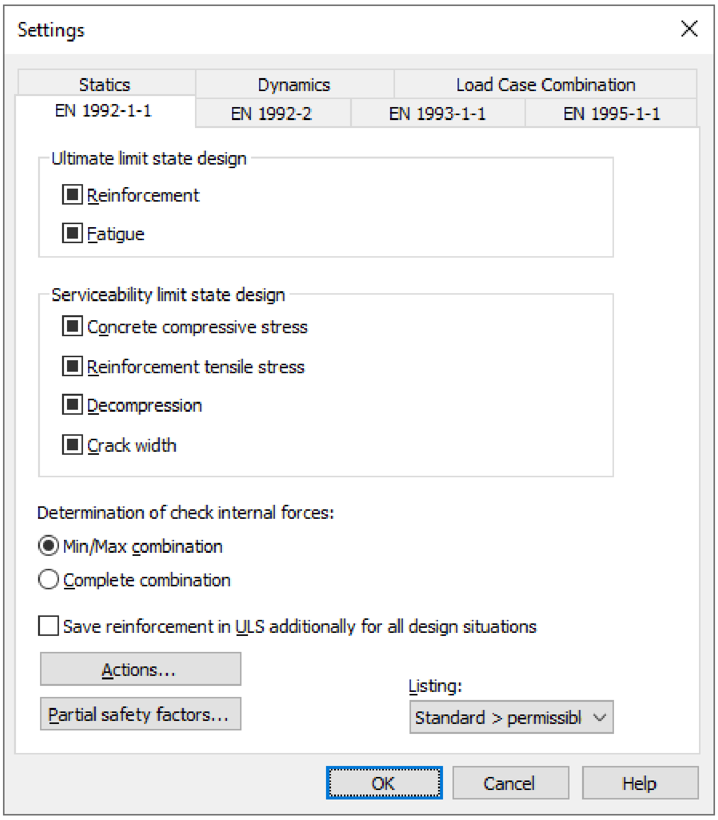
The coefficients used to take into account the variation of prestressing force are defined in EN 1992-1-1 depending on the prestressing type. In the dialog, values are suggested according to Chapter 5.10.9 (1)P for subsequent bond. In the actions, design situations are declared as construction stage or final state. The defined variation coefficients are taken into account for the effects from internal prestressing in the following checks:

- Decompression and concrete compressive stress check.
- Minimum reinforcement for crack width limitation.
- Crack width check.

Regarding the effects from external prestressing, the variation coefficients correspond to $r_{sup} = r_{inf} = 1$.

Analysis Settings

The *EN 1992-1-1* dialog page can be opened using the *Settings* function in the *Analysis* menu.



Check selection

When selecting checks, the following cases are to be distinguished:

or The check is performed according to the settings in the section dialog (see Section inputs).

or The check is performed for all sections of the structure.

or The check is performed for no sections of the structure.

Corresponding section settings are bundled as follows:

Reinforcement	Bend and longitudinal force Lateral force Torsion Robustness Shear joint
Crack width	Minimum crack reinforcement Calculation of the crack width

An overview of the checks can be accessed using the *Design Settings* function in the *EN 1992-1-1 Design* folder of the database.

Determination of the check internal forces

- *Min/Max combination*
  The minimum and maximum values are determined for each component of the internal forces in compliance with the combination rule. Together with the associated values, these form the check internal forces.
- *Complete combination*
  To determine the check internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The calculation effort increases exponentially with the number of inclusive load cases.

The differences between the two methods are explained in more detail in the section *Check internal forces*.

Save reinforcement in ULS additionally for all design situations

In addition to the maximum required ultimate limit state reinforcement, the reinforcement is saved separately for each design situation in the ultimate limit state.

Actions...
Open the dialog for describing actions.

Partial safety factors...
Open the dialog for modifying partial safety factors.

Listing
- *No*: No log is generated by the checking program.
- *Standard*: Log with tabular output of results.
- *Detailed*: Additional output of the decisive combination internal forces at the check locations.
- *Standard > permissible*: Standard log limited to check locations where the permissible limit values are exceeded.
- *Detailed > permissible*: Detailed log limited to check locations where the permissible limit values are exceeded.

Single Design

The single design function allows you to analyze individual sections independently of the global system using predefined internal forces. The calculation is carried out from the opened input table via the *Single Design* item in the *Analysis* menu or the *Print Preview* function.

Enter the information listed below in the *Single Design* table in the *EN 1992-1-1 Design* folder of the database or the folders of the national variants.

Section

Number of the section to be designed. Both polygon and composite sections can be designed.

Combination

Design situation according to EN 1992-1-1, Table 2.1.

- *0*: Permanent and temporary design situation
- *1*: Accidental design situation

Nsd, Mysd, Mzsd

Internal forces being designed. The internal forces refer to the centroid in polygon sections or the section zero point in composite sections.

Mode

- *Standard*: Standard design mode for bending with normal force throughout the load area. Reinforcement will be calculated in the tensile section to the greatest degree possible.
- *Symmetrical*: Design for symmetrical reinforcement. As opposed to the standard mode, all of the reinforcement layers will be increased if a reinforcement increase is necessary. The predefined relationships between the reinforcement layers will not be affected.
- *Compression member*: For compression members a symmetrical design is carried out taking into account the minimum reinforcement according to Chapter 9.5.2 (2).
- *Strains*: Determine strain state for existing reinforcing steel layers.
- *Strains SLS*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. In the compression zone, a linear strain-stress curve of the concrete with the gradient $\tan\alpha = E_{cm}$ is used.
- *Strains SLS2*: Determine strain state in the serviceability limit state for existing reinforcing steel layers. A nonlinear strain-stress curve of the concrete is used as shown in Figure 3.2. Note that a horizontal progression is assumed for strains exceeding ε_{c1}.
- *Load bearing capacity*: Determination of the load bearing capacity. All internal forces are increased up to the ultimate limit state, taking into account the existing reinforcing steel layers.
- *Maximum bending moment My*: Determination of the maximum bearable bending moment M_y. The moment M_y is increased up to the ultimate limit state, taking into account the other internal forces and the existing reinforcing steel layers.
- *Inactive*: Design disabled.

OENORM B 1992-1-1:
In the modes *SLS* and *SLS2* the stress increase of the prestressing steel layers is determined according to Eq. (13AT) with the bond coefficient ξ_1 specified for the section to be checked.

Punching Shear Check

When you select a check node, the key data for the checks is displayed in a dialog field. This dialog is divided into three pages.

1a. Input data, column

The column forms *rectangle* and *round* with the locations *internal*, *edge parallel to x*, *edge parallel to y* and *corner* are available. When you enter a new column, the program will suggest the dimensions of existing columns. The edge distances a_x and a_y are used to calculate the perimeters u_i of the check sections for columns near to an edge or a corner.

DIN EN 1992-1-1:
In addition, the check locations *Wall end*, *Wall corner Fig. NA.6.12.1 (top)* or *Wall corner Fig. NA.6.12.1 (bottom)* can be selected.

In the case of *Wall end*, the check sections are determined according to the middle part of Figure NA.6.12.1 and, in the case of *Wall corner Fig. NA.6.12.1 (top)*, according to the upper part of Figure NA.6.12.1. The larger of the two entered dimensions b_x, b_y is taken as value a, the smaller as value b and from this the dimensions a_1, b_1 are calculated as shown in the figure.

In the case of *Wall corner Fig. NA.6.12.1 (bottom)*, the check sections are determined according to the bottom part of Figure NA.6.12.1. Accordingly, the dimensions b_x, b_y are not entered by the user in the dialog, but are calculated with 1.5·d and used for the check section.

OENORM B 1992-1-1:
In addition, the check locations *Wall end* or *Wall corner* can be selected. The check sections are determined according to Figure 3AT. The larger of the two entered dimensions b_x, b_y is taken as value a, the smaller as value b and from this the dimensions a_1, b_1 are calculated as shown in the figure.

1b. Input data, slab
This section shows the material properties, the existing reinforcement (a_{sx}, a_{sy}) as well as additional coefficients for calculating punching shear resistances:

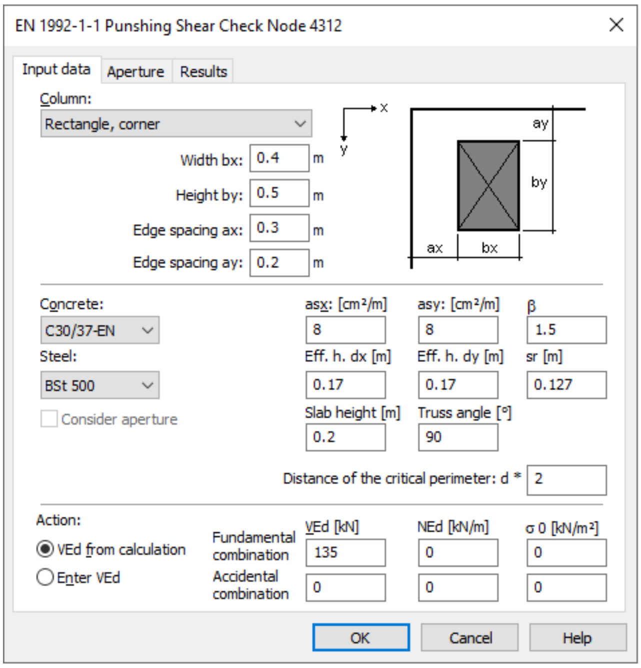
β	load increase factor for taking into account eccentric load introduction
s_r	radial distance of the punching reinforcement rows [m]
d	average value of the effective heights d_x and d_y in orthogonal directions [m]
d *	factor for the distance of the critical perimeter related to the effective height d

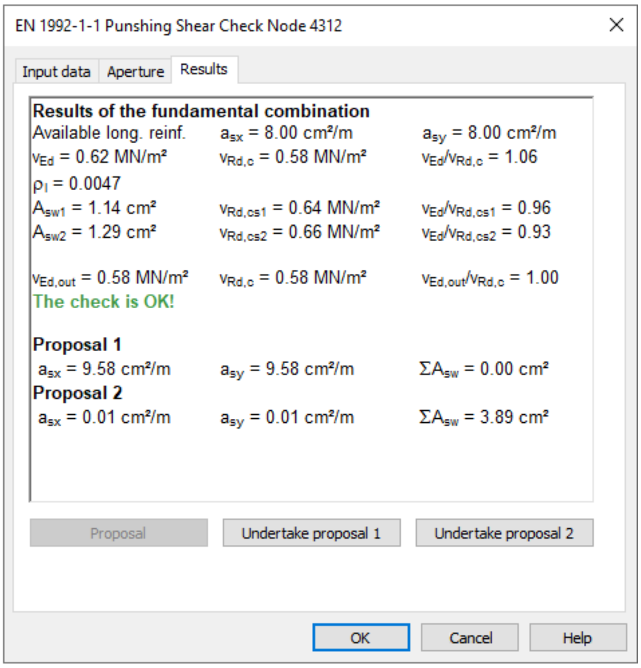
1c. Input data, action
The action V_{Ed} can either be added as a support force from a previous design according to EN 1992-1-1 or defined directly. All medium soil pressures σ_0 lower the design value of the lateral force within the area of the decisive perimeter. The medium longitudinal forces N_{Ed} are used to calculate the normal concrete stress.

2. Aperture
This dialog page is used to define the geometry and location of an opening.

3. Results
This dialog page shows the calculated punching shear resistances, the necessary punching shear reinforcement (if applicable) and the minimum bending reinforcement (if nationally relevant). You can call up an improved bending reinforcement by clicking the *Proposal* button.

Example



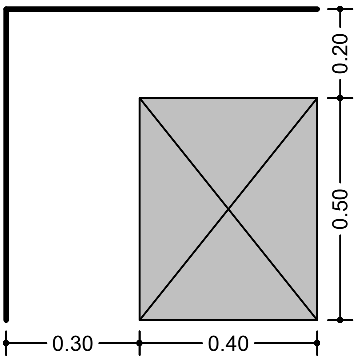


Punching shear check node 4312

The check is performed according to EN 1992-1-1:2004/A1:2014.

1. Measurements, situation and material

Rectangular column with width b_x = 0.40 m and height b_y = 0.50 m
Situation: Corner column; Edge spacing a_x = 0.30 m; Edge spacing a_y = 0.20 m; β = 1.50



Critical perimeter u_1 = 1.93 m (Distance = 0.34 m); A_1 = 1.06 m²

Slab height h = 0.200 m
Effective height of the slab d_x = 0.170 m; d_y = 0.170 m; $d = (d_x + d_y) / 2$ = 0.170 m
Available longitudinal reinforcement a_{sx} = 8.00 cm²/m; a_{sy} = 8.00 cm²/m
Truss angle α = 90.0°

Concrete: C35/45-EN	f_{ck} = 30.00 MN/m²	α_{cc} = 1.00
	γ_c = 1.50	$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c$ = 20.00 MN/m²
Reinforce.: BSt 500	f_{ck} = 500.00 MN/m²	γ_s = 1.15
	$f_{yd} = f_{yk} / \gamma_s$ = 434.78 MN/m²	

2. Action from fundamental combination

V_{Ed} = 135.00 kN N_{Ed} = 0.00 kN/m σ_0 = 0.00 kN/m²

$v_{Ed} = \beta \cdot V_{Ed} / (u_i \cdot d)$ (6.38)

with $u_i = u_1$

v_{Ed} = 0.62 MN/m²

3. Punching resistance without punching reinforcement

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \qquad (6.47)$$

$C_{Rd,c} = 0.18 / \gamma_c$

with $C_{Rd,c}$ = 0.12 k = 2.00

ρ_l = 0.0047 f_{ck} = 30.00 MN/m²

k_1 = 0.10 $\sigma_{cp} = -N_{Ed} / h$ = 0.00 MN/m²

v_{min} = 0.54 MN/m²

$v_{Rd,c}$ = 0.58 MN/m²

$v_{Ed} / v_{Rd,c}$ = 1.06 > 1 Punching reinforcement is required!
$v_{Ed,0} = \beta \cdot V_{Ed} / (u_0 \cdot d)$ = 2.34 < $v_{Rd,max}$ = 4.22 MN/m² (6.53)

with u_0 = 0.51 m

4. Punching reinforcement (normal)

$$A_{sw} = \frac{(v_{Ed} - 0{,}75 \cdot v_{Rd,c})}{1{,}5 \cdot (d / s_r) \cdot f_{ywd,ef} \cdot (1/(u_1 \cdot d))} \qquad (6.52)$$

$$A_{sw,i,min} = 0{,}08 \cdot \sqrt{f_{ck}} / f_{yk} \cdot \frac{s_r \cdot u_{cont,i}}{1{,}5} \qquad (9.11)$$

with v_{Ed} = 0.62 MN/m² $v_{Rd,c}$ = 0.58 MN/m²

s_r = 0.12 m $f_{ywd,ef}$ = 292.50 MN/m²

f_{ck} = 30.00 MN/m² f_{yk} = 500.00 MN/m²

Row 1: Distance = 0.09 m; $u_{cont,1}$ = 1.53 m; $A_{sw,1}$ = 1.08 cm² > $A_{sw,1,min}$ = **1.08 cm²**

Row 2: Distance = 0.21 m; $u_{cont,2}$ = 1.72 m; $A_{sw,2}$ = 0.96 cm² > $A_{sw,2,min}$ = **1.21 cm²**

External perimeter according to Equ. (6.54) and Fig. 6.22 A
$u_{out} = \beta \cdot V_{Ed} / (v_{Rd,c} \cdot d)$ = 2.05 m
Distance = 0.42 m
The outermost reinf. row is placed at a spacing of 0.21 m ≤ 1.5 · d = 0.26 m. The check is OK!

Maximal load bearing capacity with punching reinforcement acc. to Eq. (6.52)
$v_{Ed} = 0.62 \leq k_{max} \cdot v_{Rd,c} = 1.50 \cdot 0.58 = 0.87$. The check is OK!

Prestressed Structures

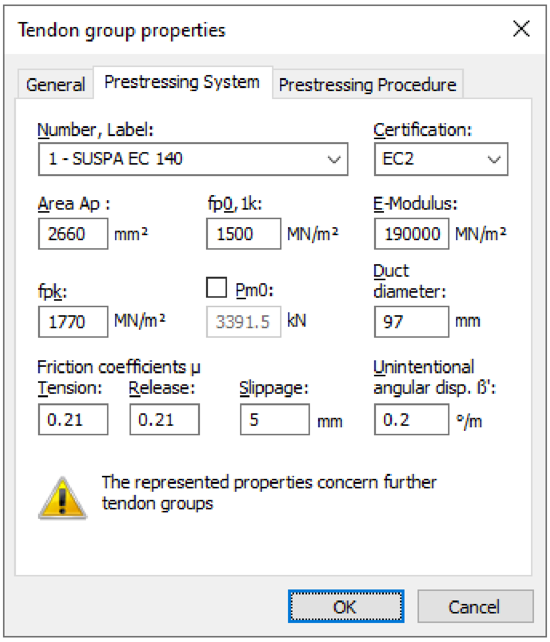
Internal Prestressing

For internal prestressing, the tendon groups as well as the prestressing system and procedures are entered using the *Prestressing* function of the *Structure* menu. To include them in the FEM calculation, you then need to define a load case of the *Prestressing* load type.

Prestressing with bond and prestressing without bond are differentiated in the section inputs and the specifications for the *Creep and shrinkage* load case. For prestressed components with subsequent bond the tendons can be set ungrouted for the respective design situation in the action dialog.

Prestressing System

The prestressing system combines typical properties that are then assigned to the tendon groups using a number.



Number, Label

Number and name of the prestressing system. The option <Database> enables to load or to store properties by use of the file *Igraph.dat*.

Certification

- DIN 1045-1
- DIN 4227
- EC2
- OENORM
- SIA 262

By selection of the certification, the prestressing force P_{m0} is determined according to the standard.

Area Ap

Section area A_p of a tendon [mm²].

ßs, ß02

Yield strength or $\beta_{0.2}$ limit of the prestressing steel according to DIN 4227 [MN/m²].

fp0,1k

Characteristic value of the 0.1% strain limit of the prestressing steel per DIN 1045-1, OENORM, SIA 262 and EC2 [MN/m²].

E-Modulus

E-modulus of the prestressing steel [MN/m²].

ßz

Tensile strength of the prestressing steel according to DIN 4227 [MN/m²].

fpk

Characteristic value of the tensile strength of the prestressing steel per DIN 1045-1, OENORM, SIA 262 and EC2 [MN/m²].

Pm0

The permissible prestressing force of a tendon [kN] that corresponds to the selected certification is displayed where the minimum of the two possible values is decisive. After releasing the input field, a different prestressing force can be defined.

Certification as per DIN 1045-1:
$P_{m0} = A_p \cdot 0.85 f_{p0,1k}$ or $A_p \cdot 0.75 f_{pk}$ according to DIN 1045-1, Eq. (49).

Certification as per DIN 4227:
$P_{m0} = A_p \cdot 0.75 ß_s$ or $A_p \cdot 0.55 ß_z$ according to DIN 4227-1, Tab. 9, Row 65.

Certification as per EC2:
$P_{m0} = A_p \cdot 0.85 f_{p0,1k}$ or $A_p \cdot 0.75 f_{pk}$ according to EN 1992-1-1, Eq. (5.43).

Certification as per OENORM:
$P_{m0} = A_p \cdot 0.80 f_{p0,1k}$ or $A_p \cdot 0.70 f_{pk}$ according to OENORM B 4750, Eq. (4) and (5), and OENORM B 1992-1-1, Chapter 8.9.6.

Certification as per SIA 262:
$P_{m0} = A_p \cdot 0.7 f_{pk}$ according to SIA 262, Eq. (22), Chapter 4.1.5.2.2.

Duct diameter

Is used for the decompression check according to the European standard and for beam tendons to calculate the net section values [mm].

Friction coefficients

Friction coefficients μ for prestressing and release.

Slippage

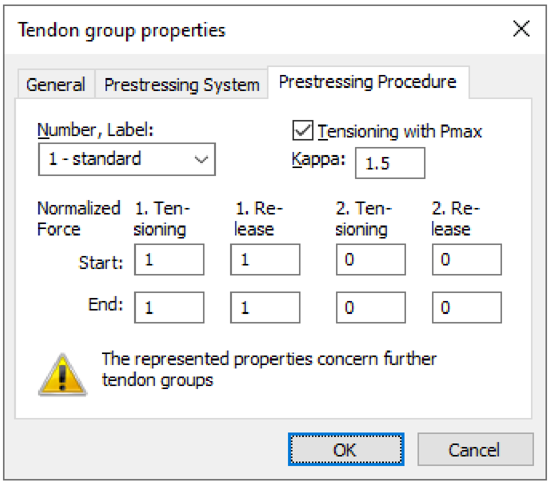
Slippage at the prestressing anchor [mm].

Unintentional deviation angle ß'

Unintentional deviation angle of a tendon [°/m].

Prestressing Procedure

The prestressing procedure differentiates between the start and end of the tendon group. The size of the maximum prestressing force is determined by factors regarding the permissible prestressing. In general, this is P_{m0} (see *Prestressing system*). Using the factor specified for the release, the maximum prestressing force remaining in the tendon group is defined with respect to P_{m0}. The prestressing force that remains at the prestressing anchor is calculated from this by the program. The resulting prestressing involves immediate losses due to friction and slippage, but not due to the elastic deformations of the concrete and the short-term relaxation. Each prestressing anchor can be prestressed and released twice. The prestressing procedures are numbered.



Number, Label
Number and name of the prestressing procedure.

Tensioning with Pmax
Selecting this check box causes the factors for tensioning correspond to the maximum force P_{max} for tendons certified according to DIN 1045-1 or EC2 (see the following example).

Kappa
If tensioning with P_{max} is selected, the permissible maximum force is calculated using the allowance value κ to ensure there is an overstressing reserve.

1. Tensioning
Factor relating to P_{m0} or P_{max} for the prestressing force at the tie at the 1st instance of tensioning.

1. Release
Factor relating to P_{m0} for the maximum remaining prestressing force at the 1st release. '0': no release!

2. Tensioning
Factor relating to P_{m0} or P_{max} for the prestressing force at the tie for the 2nd tensioning. '0': no 2nd tensioning!

2. Release
Factor relating to P_{m0} for the maximum remaining prestressing force at the 2nd release. '0': no 2nd release!

The prestressing force curve is determined in the following sequence:
- Tensioning and release at the start,
- Tensioning and release at the end,
- Slippage at the start,
- Slippage at the end.

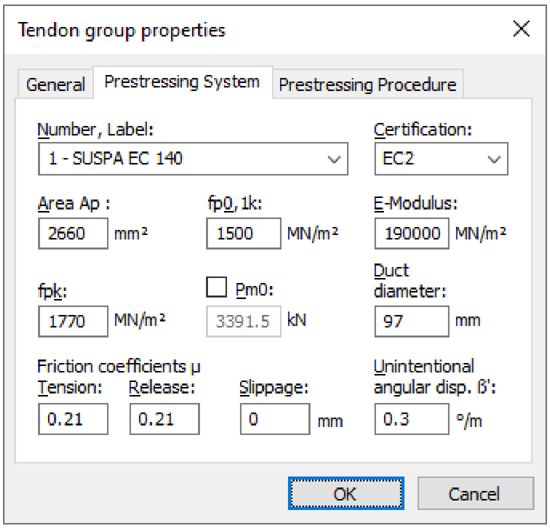
The differences between tensioning with P_{m0} and P_{max} are described in the following examples.

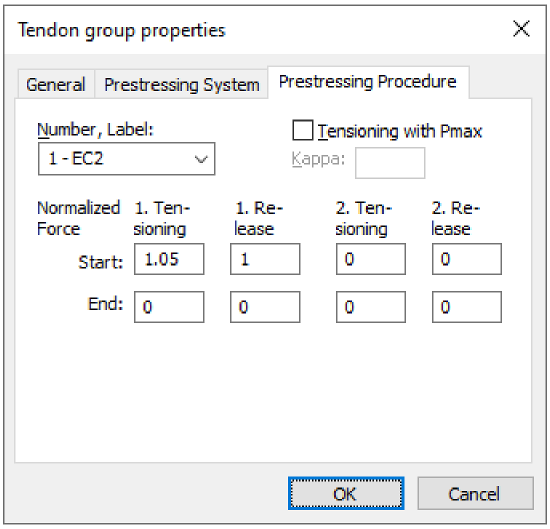
The user is responsible for checking the permissibility of the maximum force during the stressing process.

Examples for Prestressing Procedures According to EC2

Tensioning with P_{m0}

The mode of action of the factors *Tensioning* and *Release* can be clarified using the example of an St 1570 / 1770 single tendon with prestressing anchor at the tendon start certified according to EC2.





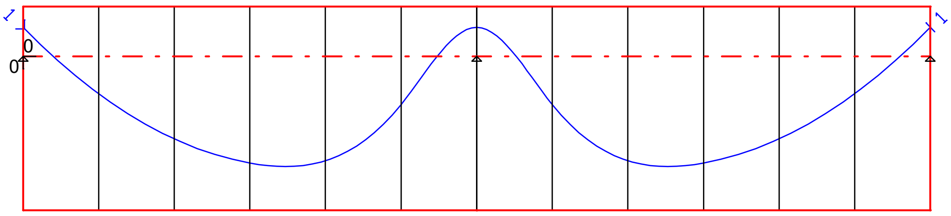
The permissible prestressing forces ar defined by:

$P_{max} = min(A_p \cdot 0.80\, f_{pk}\, ,\, A_p \cdot 0.90\, f_{p0.1k}) = 3591.0$ kN

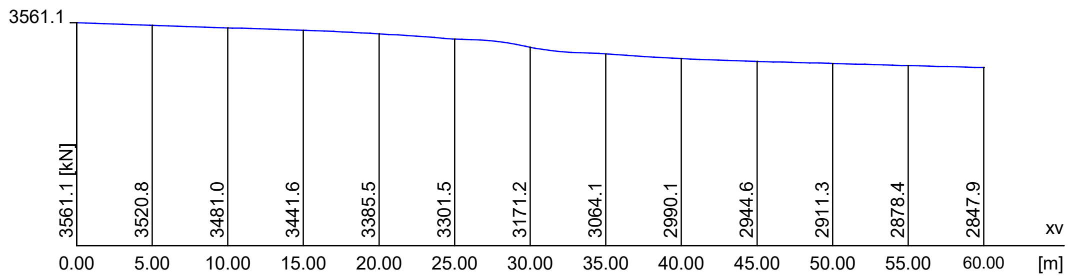
$P_{m0} = min(A_p \cdot 0.75\, f_{pk}\, ,\, A_p \cdot 0.85\, f_{p0.1k}) = 3391.5$ kN

The first prestressing force curve of the following illustration results after overstressing with 5% using a factor of 1.05 relating to P_{m0}, i.e. the maximum prestressing force is 3561.1 kN < P_{max}.

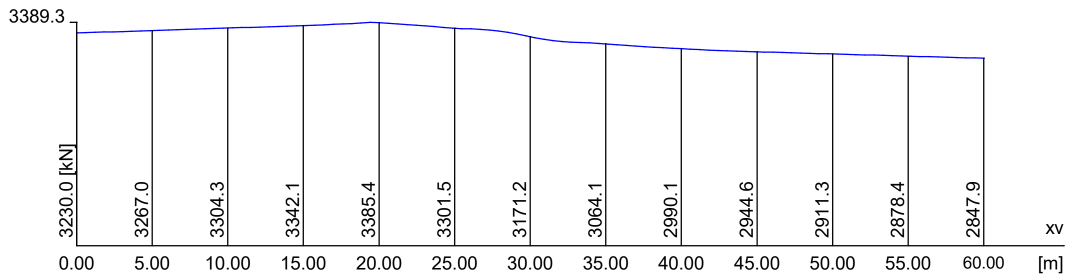
The second prestressing force curve results after tensioning and release with the factors 1.05 and 1.0, i.e. the maximum prestressing force that remains in the tendon after it is fixed into place is 3389.3 kN < P_{m0}.



Single tendon, 10 times superelevated



Prestressing force curve after the 1st tensioning with a factor of 1.05



Prestressing force curve after the 1st release with a factor of 1.0

Potential slippage was not taken into account here to illustrate the effects described above. Slippage would result in an additional variation of the prestressing force curve. A second prestressing and release procedure would have similar effects. The same holds true for prestressing and release at the tendon end.

Tensioning with P_{max}

For tendons with certification as per DIN 1045-1 and EC2 the maximum force applied to the tendon during the stressing process is determined with the smaller of the following values:

$$P_{max} = A_p \cdot 0.80\, f_{pk}\, e^{-\mu\gamma(\kappa-1)} \text{ or } A_p \cdot 0.90\, f_{p0.1k}\, e^{-\mu\gamma(\kappa-1)}$$

DIN 1045-1 rep. Book 525, Chapter 8.7.2
DIN TR 102, Chapter 4.2.3.5.4 (2)*P
DIN EN 1992-1-1, Chapter 5.10.2.1 (NA.3)

with

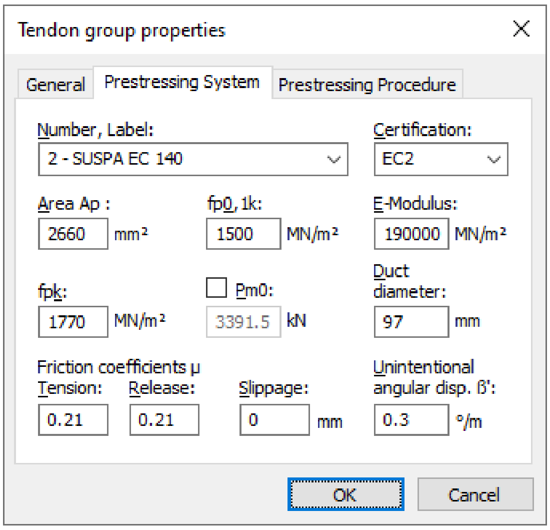
μ Friction coefficient according to the general certification from the building authorities.

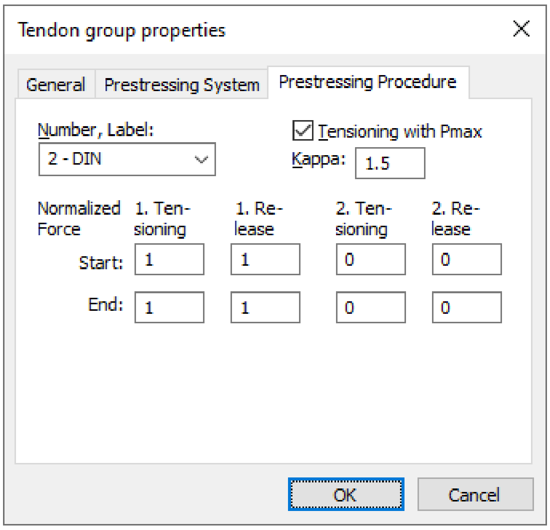
γ $\Phi + k{\cdot}x$
Φ = sum of planned deviation angle over the length x,
k = unintentional deviation angle per unit of length (ß' in the dialog),
x = the distance between the prestressed anchor and the fixed anchor in the case of one-sided prestressing or the influence length of the respective anchor in the case of two-sided prestressing.

κ Allowance value for ensuring an overstressing reserve with $1.5 \le \kappa \le 2$ for tendons with supplemental bond according to the German standard and $\kappa = 1$ for all other cases.

The program uses the specified allowance value κ to determine the maximum permissible value P_{max}. The influence length x is assumed to be the tendon length for one-sided prestressing or simply half of the tendon length for two-sided prestressing.
In this setting the overstressing factor refers to P_{max}, which means the value 1.0 is used to select the maximum planned force according to the German standard.
The release factor continues to refer to P_{m0}. Setting the value to 1.0 also assures that the force remaining in the tendon after it fixed into place is within the permissible range.

Using an St 1570 / 1770 single tendon prestressed on both sides with certification as per EC2, the prestressing force curve is illustrated for a value of κ = 1.5. Slippage is ignored for the sake of simplicity.



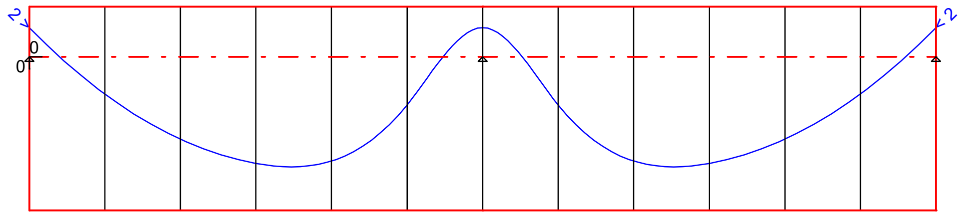


The program will determine the permissible prestressing forces as follows:

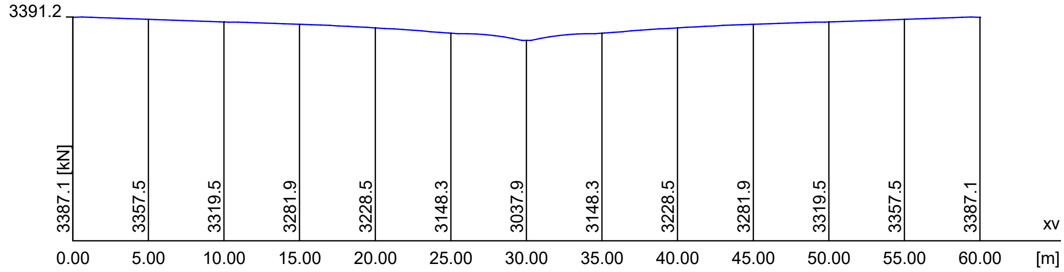
$$P_{max} = e^{-\mu\gamma(\kappa-1)} \cdot min(A_p \cdot 0.80\, f_{pk}\, ,\, A_p \cdot 0.90\, f_{p0.1k}) = 0.9457 \cdot 3591 = 3395.9 \text{ kN}$$

$$P_{m0} = min(A_p \cdot 0.75\, f_{pk}\, ,\, A_p \cdot 0.85\, f_{p0.1k}) = 3391.5 \text{ kN}$$

The maximum force P_{max} is automatically maintained with a tensioning factor of 1.0. As shown in the following force curve, 3391.2 kN remain in the tendon after it is fixed into place. Thus the limit P_{m0} is also observed.



Single tendon, 10 times superelevated



Prestressing force curve after tensioning and release

If the force calculated during prestressing is less than the value during release, then the program will make sure that the smaller value is not exceeded after the component is fixed into place.

External Prestressing, Mixed Construction

External prestressing can be taken into account by entering the external forces directly in the program. For mixed construction, the additional tendons in a bond must be entered as described above.

Variation of Prestressing

For checks in the ultimate limit state, the following applies for the design value of the prestressing force according to EN 1992-1-1, Chapter 5.10.8 (1):

$$P_{d,t}(x) = \gamma_P \cdot P_{m,t}(x)$$

with

$P_{m,t}(x)$ Mean value of prestressing force at time t and location x including prestressing losses from friction, slippage, creep, shrinkage and relaxation.

γ_P Partial safety factor of prestressing force, $\gamma_P = 1$ as specified in Chapter 2.4.2.2 (1).

In the serviceability limit state, two characteristic values for the prestressing force are defined in Chapter 5.10.9 (1):

$P_{k,sup} = r_{sup} \cdot P_{m,t}(x)$ Upper characteristic value according to Equation (5.47).

$P_{k,inf} = r_{inf} \cdot P_{m,t}(x)$ Lower characteristic value according to Equation (5.48).

The variation coefficients for internal prestressing are defined separately for construction stages and final states. They are used in the following checks:

- Decompression and concrete compressive stress check.
- Minimum reinforcement for crack width limitation.
- Crack width check.

Regarding the effects from external prestressing, the variation coefficients correspond to $r_{sup} = r_{inf} = 1$.

For internal prestressing, the recommended country-specific values are:

- For tendons with immediate bond or without bond:
  $r_{sup} = 1.05$ and $r_{inf} = 0.95$.

- For tendons with subsequent bond:
  $r_{sup} = 1.10$ and $r_{inf} = 0.90$.

OENORM B 1992-1-1:

- For tendons with immediate bond or without bond:
  $r_{sup} = r_{inf} = 1.0$.

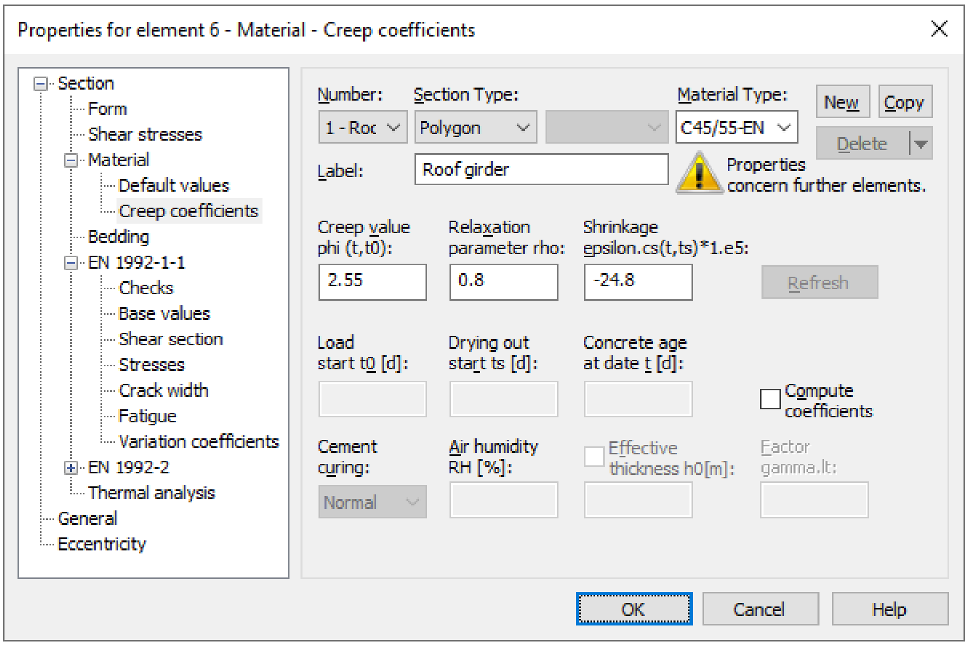
- For tendons with subsequent bond:
  $r_{sup} = 1.05$ and $r_{inf} = 0.95$.

BS EN 1992-1-1:

$r_{sup} = r_{inf} = 1.0$ generally applies.

Creep and Shrinkage

Similar to prestressing, creep and shrinkage are taken into account by specifying a corresponding load case (*Creep and shrinkage* load type) in the FEM calculation. Besides the creep-generating continuous load case, you also need to specify whether the internal forces relocation between concrete and prestressing steel is to be taken into account. This option is only useful in the case of tendons with bond.
The decisive creep and shrinkage coefficients for calculating the *Creep and shrinkage* load case are entered in the section dialog. Alternatively, you can also use this dialog to calculate the coefficients according to Chapter 3.1.4 with Annex B.



The program determines concrete creep and shrinkage based on a time-dependent stress-strain law developed by Trost.

$$\sigma_b(t) = \frac{E_b}{1+\rho\cdot\varphi}\left(\varepsilon_b(t) - \varphi\cdot\varepsilon_{b,0} - \varepsilon_{b,S}\right)$$

Explanation of the individual terms:

$\sigma_b(t)$	Concrete stress from creep and shrinkage at time t.
E_b	E-modulus of the concrete.
ρ	Relaxation coefficient according to Trost for time t (normally $\rho = 0.80$).
φ	Creep coefficient for time t.
$\varepsilon_b(t)$	Concrete strain from creep and shrinkage at time t.
$\varepsilon_{b,0}$	Concrete strain from creep-generating continuous load.
$\varepsilon_{b,s}$	Concrete strain from shrinkage.

Under consideration of these relationships, a time-dependent global stiffness matrix and the associated load vectors are constructed which, in turn, yield the internal forces and deformations of the concrete. The resulting stress changes in the prestressing steel are also determined provided they are selected in the load case. Any influence from the relaxation of the prestressing steel will be ignored in this case. According to Zilch/Rogge (2002, p. 256), this influence can be calculated separately (see following section) and combined with the changes from creep and shrinkage for all time-dependent prestressing losses:

$$\Delta\sigma_{p,csr} = \Delta\sigma_{pr} + E_p \cdot \Delta\varepsilon_{cpt}$$

with

$\Delta\sigma_{pr}$ Prestressing loss from relaxation of the prestressing steel.

$\Delta\varepsilon_{cpt}$ Concrete strain change from creep and shrinkage.

E_p E-modulus of the prestressing steel.

Relaxation of Prestressing Steel

According to EN 1992-1-1, Chapter 5.10.6, the stress change $\Delta\sigma_{pr}$ in the tendons at position x and time t due to relaxation must be taken into account in addition to the stress loss from concrete creep and shrinkage. The relaxation of the steel depends on the deformation of the concrete caused by creep and shrinkage. According to 5.10.6 (1) (b), this interaction can be taken into account in a general and approximate manner by specifying a reduction coefficient of 0.8.

The stress change $\Delta\sigma_{pr}$ can be determined for the initial stress in the tendons as a result of prestressing and quasi-continuous actions according to 5.10.6 (2). More details are provided in Chapter 3.3.2 of the standard.

The stress losses are defined in the CSR actions of the *EN 1992-1-1 actions* dialog.

DIN EN 1992-1-1:

The stress change $\Delta\sigma_{pr}$ can be determined using the specifications of the prestressing steel certification for the ratio of initial stress to characteristic tensile strength (σ_{p0}/f_{pk}). $\sigma_{p0} = \sigma_{pg0}$ may be used as the initial stress, with σ_{pg0} referring to the initial prestressing steel stress from prestressing and the permanent action.

Check Internal Forces

The calculation of load cases results in a set of internal forces for each load case at the check location (e.g. Nx, My). The check internal forces are then determined from the results of the load cases with the combination rules relevant for the ultimate limit state, fatigue and serviceability limit state. One of the following methods can be selected in the analysis settings:

- *Min/Max combination*
  The results of a load case are added to the set of internal forces with the minimum or maximum of an internal force, if this increases the amount of the extreme value. Result sets from traffic actions in which the control variable is less than the threshold 10^{-3} are not combined. The min/max combination delivers a constant number of sets regardless of the number of load cases and thus represents a particularly economical solution for the checks.
- *Complete combination*
  To determine the evidence internal forces, all possibilities of interaction of actions resulting from the combination rule are taken into account. The number of records increases exponentially with the number of inclusive load cases and can therefore result in high time and memory requirements for the checks.

For beams, design objects and axisymmetric elements, the resulting sets of internal forces are used directly in the checks. For area elements, *design internal forces* are derived from this, as will be described in more detail in the following section.

The internal forces relevant for the checks are documented in the detailed check listing. Regardless of the selection made, the results of the min/max combination are saved for the graphical representation. The load cases involved in the combination can be displayed using the *Combination information* context function.

The differences between the two combination methods mentioned before can be seen from the following example of a uniaxially stressed beam. The load cases 2, 3 and 4 shown can act simultaneously (inclusive). All safety and combination factors are assumed to be 1 for the example.

Action	Nx	My	Load case
G - permanent	-15	40	1
Q - variable	0	20	2
	5	10	3
	0	-10	4

Internal forces of the load cases

Extreme value	Nx	My	Combination
min Nx	-15	40	L1
max Nx	-10	50	L1+L3
min My	-15	30	L1+L4
max My	-10	70	L1+L2+L3

Results of min/max combination

Set	Nx	My	Combination
1	-15	40	L1
2	-15	60	L1+L2
3	-10	50	L1+L3
4	-15	30	L1+L4
5	-10	70	L1+L2+L3
6	-15	50	L1+L2+L4
7	-10	40	L1+L3+L4
8	-10	60	L1+L2+L3+L4

Results of complete combination

Design internal forces for area elements

With area elements, the design internal forces correspond to the plasticity approach from Wolfensberger and Thürlimann. This approach takes into account how much the reinforcement deviates from the crack direction. Due to the current lack of precise data regarding the combined load of reinforced concrete shell structures from bending and normal force, the design internal forces for bending and normal force are calculated independently according to the static limit theorem of the plasticity theory and then used together as the basis for the design in the two reinforcement directions. This approach should always lead to results that are on the safe side.

Depending on the type of area element and reinforcement configuration, the variants of design internal forces listed below are taken into account for the checks.

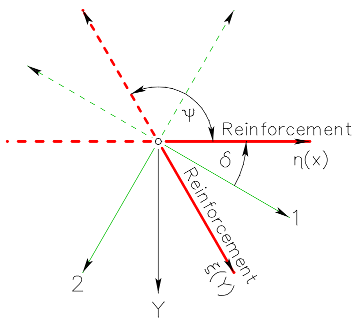
Orthogonal area reinforcement

Slabs	$m_x \pm \lvert m_{xy}\rvert$
	$m_y \pm \lvert m_{xy}\rvert$
Plain stress	$n_x \pm \lvert n_{xy}\rvert$
elements	$n_y \pm \lvert n_{xy}\rvert$
Shells	$m_x \pm \lvert m_{xy}\rvert$ and $n_x \pm \lvert n_{xy}\rvert$
	$m_y \pm \lvert m_{xy}\rvert$ and $n_y \pm \lvert n_{xy}\rvert$

Oblique area reinforcement

The bending design of slabs with oblique reinforcement assemblies is carried out according to Kuyt or Rüsch. Here the design moments are calculated with the help of the principal moments m_1, m_2 according to the equations given in Book 166 DAfStB.

For load case combinations, the calculation is based on the extreme values of m_1, m_2. For combined loads (bending and longitudinal force), both the design moments and the normal design forces are independently derived from n_1, n_2. The normal design forces are then used together as the basis for the design. This should also result in an upper limit for the load.



Coordinate systems

Extreme values (principal bending moments):

$$m_{1,2} = \frac{1}{2}\cdot(m_x + m_y) \pm \frac{1}{2}\sqrt{(m_x - m_y)^2 + 4m_{xy}^2}$$

with $m_1 \geq m_2$

The angle δ assigned to m_1 is:

$$\tan\delta = \frac{2\cdot m_{xy}}{(m_x - m_y) + \sqrt{(m_x - m_y)^2 + 4\cdot m_{xy}^2}}$$

Design moments:

$$m_\eta = \frac{1}{\sin^2\psi}\left[m_1 \sin^2(\delta+\psi) + m_2\cos^2(\delta+\psi) \pm \left| m_1 \sin\delta\,\sin(\delta+\psi) + m_2\cos\delta\,\cos(\delta+\psi)\right|\right]$$

$$m_\xi = \frac{1}{\sin^2\psi}\left[m_1 \sin^2\delta + m_2\cos^2\delta \pm \left| m_1 \sin\delta\,\sin(\delta+\psi) + m_2\cos\delta\,\cos(\delta+\psi)\right|\right]$$

The formulas apply accordingly for the normal design forces.

Checks in the Ultimate Limit States

The following checks are available:

- Bending with or without normal force or normal force only (EN 1992-1-1, Chapter 6.1).
- Minimum reinforcement against failure without warning (Chapter 5.10.1 (5)P and 9.2.1.1).
- Lateral force (Chapter 6.2).
- Torsion and combined stressing (Chapter 6.3).
- Shear joint (Chapter 6.2.5).
- Punching shear (Chapter 6.4).

Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.4.3, the following combinations are taken into account in the ultimate limit states:

- For the combination of the permanent and temporary design situation either Equation (6.10) or the most unfavorable equation from (6.10a) and (6.10b) is permitted.

$$\sum_{j\geq 1}\gamma_{G,j}\cdot G_{k,j}\text{"+"}\;\gamma_P\cdot P\text{"+"}\;\gamma_{Q,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \qquad (6.10)$$

$$\sum_{j\geq 1}\gamma_{G,j}\cdot G_{k,j}\text{"+"}\;\gamma_P\cdot P\text{"+"}\;\gamma_{Q,1}\cdot\psi_{0,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \qquad (6.10a)$$

$$\sum_{j\geq 1}\xi_j\cdot\gamma_{G,j}\cdot G_{k,j}\text{"+"}\;\gamma_P\cdot P\text{"+"}\;\gamma_{Q,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \qquad (6.10b)$$

For the coefficient ξ the value of $\xi = 0.85$ results from Table A.1.2(B).

DIN EN 1990, OENORM B 1990:
Equation (6.10) is used for the combination.

SS EN 1990 (EKS 11):
Equations (6.10a) and (6.10b) apply with following modifications:

$$\sum_{j\geq 1}\gamma_d\cdot\gamma_{G,j}\cdot G_{k,j}\text{"+"}\;\gamma_P\cdot P \qquad (6.10aSS)$$

$$\sum_{j\geq 1}\xi_j\cdot\gamma_d\cdot\gamma_{G,j}\cdot G_{k,j}\text{"+"}\;\gamma_P\cdot P\text{"+"}\;\gamma_d\cdot\gamma_{Q,1}\cdot Q_{k,1}\text{"+"}\sum_{i>1}\gamma_d\cdot\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \qquad (6.10bSS)$$

Assuming reliability class 3, factor γ_d is set to 1. (see Section A, Article 11 and 14). The coefficient ξ is set to the value of $\xi = 0.89$.

BS EN 1990:
The coefficient ξ in Equation (6.10b) is set to the value of $\xi = 0.925$.

- Combination for accidental design situations

$$\sum_{j\geq 1}G_{k,j}\text{"+"}\;P\text{"+"}\;A_d\text{"+"}\;(\psi_{1,1}\,\text{or}\,\psi_{2,1})\cdot Q_{k,1}\text{"+"}\sum_{i>1}\psi_{2,i}\cdot Q_{k,i} \qquad (6.11b)$$

$\psi_{1,1}\cdot Q_{k,1}$ is used by the program for this combination.

OENORM B 1990-1:
$\psi_{2,1}\cdot Q_{k,1}$ is decisive.

- Combination for design situations caused by earthquakes

$$\sum_{j\geq 1}G_{k,j}\text{"+"}\;P\text{"+"}\;A_{Ed}\text{"+"}\sum_{i\geq 1}\psi_{2,i}\cdot Q_{k,i} \qquad (6.12b)$$

For each combination you can define different design situations for the construction stages and final states. When conducting the check, the extreme value deriving from all combinations and situations is decisive.

Stress-Strain Curves

The following characteristics are used for section design:

- Concrete: Parabola-rectangle diagram according to EN 1992-1-1, Figure 3.3. Note that the design value for concrete compressive strength f_{cd} in Equation (3.15) is defined as $f_{cd}=\alpha_{cc} \cdot f_{ck}/\gamma_c$ with $\alpha_{cc}=1$ for normal concrete and $\alpha_{cc}=0.85$ for lightweight concrete.

  DIN EN 1992-1-1:

  α_{cc} = 0.85 for normal concrete and α_{cc} = 0.75 for lightweight concrete.

  SS EN 1992-1-1:

  α_{cc} = 1 for normal and lightweight concrete.

  BS EN 1992-1-1:

  According to NA to 3.1.6 (1)P conservatively, $\alpha_{cc} = 0.85$ is always assumed for normal concrete and lightweight concrete.

- Reinforcing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.8, with rising upper branch, where the maximum stress is assumed to be $k \cdot f_{yk} / \gamma_s$ with k = 1.05 as per Table C.1, class A.

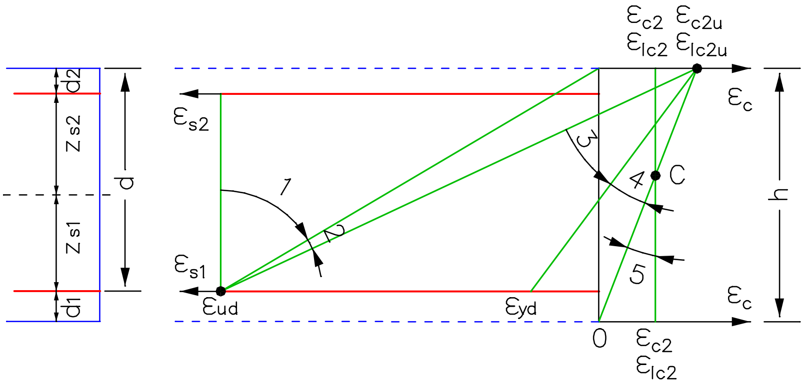
  DIN EN 1992-1-1:

  The maximum stress is assumed to be $1.05 \cdot f_{yk} / \gamma_s$ for ductility class A according to DIN 488-1.

- Prestressing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.10, with horizontal upper branch according to Chapter 3.3.6 (7) of the standard and a maximum stress of $f_{pd} = f_{p;0,1k} / \gamma_s$.

Design for Bending With or Without Normal Force or Normal Force Only

The design for longitudinal force and bending moment is performed according to EN 1992-1-1, Chapter 6.1. The reinforcement required for each internal force combination at the reinforced concrete section is determined iteratively based on the formulation of equilibrium conditions as well as the limit strain curve depicted in the illustration below. The final result is derived from the extreme value of all calculated reinforcements.



Strain areas for the design with $\varepsilon_{ud} = 0.9\ \varepsilon_{uk}$ and $\varepsilon_{uk} = 0.025$ as per Table C.1.

DIN EN 1992-1-1:

$\varepsilon_{ud} = 0.025$

You can control the result of the design by specifying the reinforcement geometry and choosing one of three design modes. For sections subject to a compressive normal force, the minimum eccentricity defined in Chapter 6.1 (4) is taken into account. Concrete compression according to Chapter 6.1 (5) cannot be checked.

Standard Mode

This is the standard design mode for bending with longitudinal force throughout the entire load area. Reinforcement will be calculated in the tensile section to the greatest degree possible. Given ratios between certain reinforcement layers in the tension or compression zone are maintained as far as possible, unless this is deselected in the design specifications. The procedure in strain areas 4 and 5 is the same as with symmetrical design. The required transverse reinforcement of slab as per Section 9.3.1.1 (2) is considered during design according to user specification. However, the provision for horizontal reinforcement of walls as per Section 9.6.3 (1) is not taken into account.

DIN EN 1992-1-1:
The referenced compressive zone height x_d / d is limited according to Chapter 5.4 (NA.5) and NA.11.5.2 (1) as follows:

x_d / d ≤ 0.45 for concrete strength classes up to C50/60.

≤ 0.35 for concrete strength class C55/67 or higher and for lightweight concrete.

Symmetrical Mode

In contrast to the standard design, the reinforcement will be applied at all predefined locations in all strain areas, if necessary. The specified ratios between the reinforcement layers will not be affected unless this is deselected in the design specifications.

Compression member Mode

The design is performed symmetrically. In addition, the minimum reinforcement required by EN 1992-1-1, Chapter 9.5.2 (2), will be calculated:

$A_{s,min} = 0.10 \mid N_{Ed} \mid / f_{yd}$ or $0.002\ A_c$, depending on which value is greater (9.12N)

with

N_{Ed} Design value of the longitudinal force to be absorbed.

f_{yd} Design value for the reinforcing steel strength at the yield strength.

DIN EN 1992-1-1:

$A_{s,min} = 0.15\ N_{Ed} / f_{yd}$ (9.12DE)

OENORM B 1992-1-1:

$A_{s,min} = 0.13\ N_{Ed} / f_{yd} \geq 0.0026\ A_c$ (30AT)

SS EN 1992-1-1:

$A_{s,min} = 0.002\ A_c$ (Article 28)

Inclusion of tendons with bond

When designing beams and design objects, the internal forces of the concrete section is reduced by the statically determined portions which result from prestressing minus the losses from creep, shrinkage and prestressing steel relaxation (CSR). Situations prior to the grouting of the tendons are excluded. So only the restraint portions from 'P+CSR' and the external loads are contained in the remaining internal forces for the composite section. If necessary, the reinforcing steel positioned by the user will be increased until the composite internal forces can be absorbed. In the design mode *compression member* the prestressing steel area is taken into account when determining the minimum reinforcement according to Chapter 9.5.2 (2).

The position of the tendon groups in the section, the prestressing losses from CSR, the statically determined portions and the internal forces of the concrete section and the composite section are written to the detailed log.

As a separation into statically determined and undetermined shares of the internal forces from prestressing is not possible for shell structures, the prestressing is taken into account fully on the action side when designing the longitudinal reinforcement. As a result, on the resistance side only mild steel and concrete are considered whereas the strain reserves of the tendons with bond are not used.

Minimum Reinforcement Against Failure Without Warning

With respect to prestressed concrete structures, a component failure without warning may not be caused by a tendon failure according to EN 1992-1-1, Chapter 5.10.1 (5)P. The failure can be prevented by adding the minimum reinforcement as described in Chapter 9.2.1 or any other measure listed in Section (6).

The minimum reinforcement is to be dimensioned according to Chapter 9.2.1 with Equation (9.1N) (also applies to reinforced concrete components). To account for this in the program, specify a base reinforcement in the reinforcing steel description.

Alternatively, you can select the minimum reinforcement in the section dialog based on the methods specified for prestressed concrete bridges in EN 1992-2, Chapter 6.1 (109) (robustness reinforcement). This reinforcement is determined based on Equation (6.101a):

$$A_{s,min} = M_{rep} / (z_s \cdot f_{yk}) \tag{6.101a}$$

with

M_{rep} Crack moment without allowance for prestressing force and under the assumption that the edge tensile stress corresponds to f_{ctm}. According to Chapter 9.2.1.1 (4), the 1.15-fold crack moment is used for components with unbonded tendons or with external prestressing.

z_s Lever arm of the internal forces in the ultimate limit state.

According to EN 1992-2, Chapter 6.1 (110), the minimum reinforcement should be added in areas where tensile stresses in the concrete occur under the characteristic action combination. This process should take into account the statically undetermined prestressing effect and ignore the statically determined effect.

The program determines all stresses at the gross section. The statically determined prestressing effect can only be subtracted for beams and design objects. For area elements the prestress is alternatively reduced by a user-defined reduction factor. The lever arm z_s of the internal forces is assumed as $0.9 \cdot d$ for the sake of simplicity. The calculated reinforcement is evenly distributed to the reinforcement layers in the tensile zone. In the design mode *symmetrical* reinforcement is also applied to the remaining layers. This will not affect the predefined relationships between the individual reinforcement layers. For sections with mode *compression member* the robustness reinforcement is not checked because minimum reinforcement is already determined during the design for bending with longitudinal force.

DIN EN 1992-1-1:

To ensure a ductile component behavior, the above-mentioned robustness reinforcement must be added instead of the minimum reinforcement from Equation (9.1N) for components with or without prestressing. The reinforcement must be evenly distributed in the tensile zone. As it is not specified in more detail, this reinforcement is determined with the characteristic combination as described above. The option to take tendons into account is not used by the program.

SS EN 1992-1-1:

According to Article 13, method D (proofs concerning the reliability of the tendons), in combination with at least one of the other methods, should be used. The second condition can be covered by adding the minimum reinforcement as described in Chapter 9.2.1 (method A) or by use of the above-mentioned robustness reinforcement.

Surface Reinforcement

To prevent concrete spalling, a surface reinforcement may be necessary according to EN 1992-1-1, Chapter 9.2.4. For more information, refer to Annex J. The reinforcement determined in this manner can be incorporated into the program by specifying a base reinforcement in the reinforcing steel description.

OENORM B 1992-1-1:

The guidelines set forth in Annex J are not normative.

Design for Lateral Force

Lateral force design involves determining the lateral force reinforcement and includes a concrete strut check according to EN 1992-1-1, Chapter 6.2. The following special conditions apply:

- The angle of the lateral force reinforcement is assumed to be 90°.
- The value for $\cot \Theta$ can be selected by the user within the permissible national limits of Equation (6.7N).

  DIN EN 1992-1-1:
  In the calculation, the specified value for $\cot \Theta$ is limited to the range permitted in accordance with Equation (6.7aDE) (method with load-dependent strut angle), unless the check with a constant value is selected in the section dialog. The actual effective angle of the concrete struts is logged for each check location.
- The minimum reinforcement according to Chapter 9.2.2 (5) of the standard is included in the calculated stirrup reinforcement. For areas, the minimum reinforcement as per Chapter 6.2.1 (4) will only be determined if the lateral force reinforcement is necessary for computation. For beams no minimum reinforcement is calculated for the direction with $M = Q = 0$.
- For beams and design objects, the shear design is performed separately for the Q_y and Q_z lateral forces.
- Slab and shell elements are designed for lateral force $q_r = \sqrt{(q_{x^2} + q_{y^2})}$. Depending on which has a negative effect, either the principal compressive force or principal tensile force is used for the associated longitudinal force.

  DIN EN 1992-1-1:
  If selected, the check will be carried out separately for the reinforcement directions x and y in accordance with Chapter 6.2.1 (10). In this case, the normal force in reinforcement direction is used for the associated longitudinal force. If lateral force reinforcement is necessary, it must be added from both directions.
- There is no reduction of the action from loads near supports as specified in Chapter 6.2.1 (8) of the standard.
- For beams and design objects, the decisive values of the equivalent rectangle are determined by the user independently of the normal section geometry. The coefficients for calculating the inner lever arm z based on the effective width and effective height must also be specified. For area elements, the calculation is generally performed with the lever arm $z = 0.9\ d$.
- DIN EN 1992-1-1:
  According to 6.2.3 (1), the inner lever arm is limited to the maximum value derived from $z = d - c_{v,l} - 30$ mm and $z = d - 2c_{v,l}$. Note that $c_{v,l}$ refers to the extent to which longitudinal reinforcement is laid in the concrete compressive zone.
- For beam sections with internal prestressing, the design value of lateral load-bearing capacity $V_{Rd,max}$ according to Chapter 6.2.3 (6) is determined using the nominal value $b_{w,nom}$ of the section width.
- The necessity of a lateral force reinforcement is analyzed according to Chapter 6.2.2 (1) of the standard. The special conditions listed in Sections (2) through (7) are not used in this case.
- BS EN 1992-1-1:
  The shear strength of concrete of strength classes higher than C50/60 is limited to the value of class C50/60, according to NA to 3.1.2 (2)P. The concrete compressive strength f_{cd} according to Eq. (3.15) is determined conservatively with $\alpha_{cc} = 0.85$.

The used formulas from EN 1992-1-1 that are used are listed below.

Components without computationally necessary lateral force reinforcement

$$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100\ \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}]\ b_w \cdot d \tag{6.2a}$$

with at least

$$V_{Rd,c} = (\ v_{min} + k_1 \cdot \sigma_{cp}\ )\ b_w \cdot d \tag{6.2b}$$

For lightweight concrete applies:

$$\begin{aligned} V_{lRd,c} &= [C_{lRd,c} \cdot \eta_1 \cdot k \cdot (100\ \rho_l \cdot f_{lck})^{1/3} + k_1 \cdot \sigma_{cp}]\ b_w \cdot d \\ &\geq (\eta_1 \cdot v_{l,min} + k_1 \cdot \sigma_{cp})\ b_w \cdot d \end{aligned} \tag{11.6.2}$$

where

f_{ck}, f_{lck}	is the characteristic concrete strength [N/mm²].
k	$= 1 + \sqrt{(200 / d)} \leq 2.0$ with d specified in mm.
ρ_l	$= A_{sl} / (b_w \cdot d) \leq 0.02$.
A_{sl}	is the area of the tensile reinforcement that extends at least ($l_{bd} + d$) beyond the analyzed section (see Figure 6.3).

b_w is the smallest section width in the tensile zone of the section [mm].

σ_{cp} $= N_{Ed} / A_c < 0.2 f_{cd}$ [N/mm²].

N_{Ed} is the normal force in the section due to loading or prestressing [N] ($N_{Ed} > 0$ for compression). The influence of the forced deformations on N_{Ed} can be ignored.

A_c is the entire area of the concrete section [mm²].

$V_{Rd,c}$, $V_{lRd,c}$ is the design value of the lateral force resistance [N].

η_1 is the reduction coefficient for lightweight concrete according to Eq. (11.1).

The recommended values are:

$C_{Rd,c}$ $= 0.18 / \gamma_c$ for normal concrete.

$C_{lRd,c}$ $= 0.15 / \gamma_c$ for lightweight concrete.

k_1 $= 0.15$

v_{min} $= 0.035\, k^{3/2} \cdot f_{ck}^{1/2}$ for normal concrete. (6.3N)

$v_{l,min}$ $= 0.028\, k^{3/2} \cdot f_{lck}^{1/2}$ for lightweight concrete according to 11.6.1 (1).

DIN EN 1992-1-1:

$C_{Rd,c}$ $= C_{lRd,c} = 0.15 / \gamma_c$

k_1 $= 0.12$

v_{min} $= (\kappa_1 / \gamma_c)\, k^{3/2} \cdot f_{ck}^{1/2}$

$v_{l,min}$ $= (\kappa_1 / \gamma_c)\, k^{3/2} \cdot f_{lck}^{1/2}$

with

κ_1 =0.0525 for $d < 600$ mm

=0.0375 for $d > 800$ mm

For 600 mm $< d \le 800$ mm can be interpolated.

Components with computationally necessary lateral force reinforcement

The angle θ between the concrete struts and the component axis perpendicular to the lateral force must be limited:

$$1 \le \cot\theta \le 2.5 \tag{6.7N}$$

DIN EN 1992-1-1:

$$1.0 \le \cot\theta \le (1.2 + 1.4\, \sigma_{cp} / f_{cd}) / (1 - V_{Rd,cc} / V_{Ed}) \le 3.0 \tag{6.7aDE}$$

with

$$V_{Rd,cc} = c \cdot 0.48 \cdot f_{ck}^{1/3}\, (1 - 1.2\, \sigma_{cp} / f_{cd}) \cdot b_w \cdot z \tag{6.7bDE}$$

The individual parts of which are

c $= 0.5$

σ_{cp} The design value of the concrete longitudinal stress at the level of the centroid of the section with $\sigma_{cp} = N_{Ed} / A_c$ in N/mm².

N_{Ed} The design value of the longitudinal force in the section caused by external actions ($N_{Ed} > 0$ as longitudinal compressive force).

For lightweight concrete the strut angle is to be limited to $\cot\theta = 2$ in accordance with Eq. (6.7aDE). The input value $V_{Rd,cc}$ from Eq. (6.7bDE) is to be multiplied by η_1 according to Eq. (11.1).

OENORM B 1992-1-1:

$0.6 \leq \tan\theta \leq 1.0$ (3AT)

If the section is in compression, then θ in the range

$0.4 \leq \tan\theta \leq 1.0$ (4AT)

may be selected.

SS EN 1992-1-1:
According to Article 15 and differing from Equation (6.7N), for prestressed components the condition $1.0 \leq \cot\Theta \leq 3.0$ applies.

For components with lateral force reinforcement perpendicular to the complement axis, the lateral force resistance V_{Rd} is the smaller value from

$V_{Rd,s} = (A_{sw} / s) \cdot z \cdot f_{ywd} \cdot \cot\theta$ (6.8)

and

$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot \nu_1 \cdot f_{cd} / (\cot\theta + \tan\theta)$ (6.9)

where

A_{sw}	is the section area of the lateral force reinforcement.
s	is the distance of the stirrups to each other.
f_{ywd}	is the design value for the yield strength of the lateral force reinforcement.
ν_1	is a reduction coefficient for the concrete strength when shear cracks occur.
α_{cw}	is a coefficient for taking into account the stress state in the compression chord.

The recommended values are:

ν_1	$= \nu$		
	with		
	$\nu = 0.6\ (1 - f_{ck} / 250)$	for normal concrete (f_{ck} in N/mm²)	(6.6N)
	$\nu = 0.5\ \eta_1\ (1 - f_{lck} / 250)$	for lightweight concrete (f_{lck} in N/mm²)	(11.6.6N)
α_{cw}	$= 1$	for non-prestressed components	
	$= (1 + \sigma_{cp} / f_{cd})$	for $0 < \sigma_{cp} \leq 0.25 f_{cd}$	(6.11aN)
	$= 1.25$	for $0.25 f_{cd} < \sigma_{cp} \leq 0.5 f_{cd}$	(6.11bN)
	$= 2.5\ (1 - \sigma_{cp} / f_{cd})$	for $0.5 f_{cd} < \sigma_{cp} \leq 1.0 f_{cd}$	(6.11cN)

where

σ_{cp}	is the average compressive stress in the concrete (indicated as a positive value) as a result of the design value for the normal force.

The maximum effective section area of the lateral force reinforcement $A_{sw,max}$ for $\cot\theta = 1$ is derived from:

$(A_{sw,max} \cdot f_{ywd}) / (b_w \cdot s) \leq ½\ \alpha_{cw} \cdot \nu \cdot f_{cd}$ (6.12)

The additional tensile force in the longitudinal reinforcement due to lateral force according to Eq. (6.18) is

$\Delta F_{td} = 0.5 \cdot V_{Ed} \cdot (\cot\Theta - \cot\alpha)$. (6.18)

DIN EN 1992-1-1:

ν_1	$= \eta_1 \cdot 0.75 \cdot \min(1.0;\ 1.1 - f_{ck} / 500)$
η_1	$= 1.0$ for normal concrete and as per Eq. (11.1) for lightweight concrete.
α_{cw}	$= 1.0$

Equation (6.12) is not applied.

BS EN 1992-1-1:

$\nu_1 \quad = \nu \cdot (1 - 0.5 \cos \alpha)$

The further regulations of NDP to 6.2.3 (3) are not taken into account.

Lateral force reinforcement (Standard design)

The lateral force reinforcement level is derived from Equation (9.4):

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin \alpha) \quad (9.4)$$

where

ρ_w is the reinforcement level of the lateral force reinforcement. In general, this level may not be smaller than $\rho_{w,min}$.

A_{sw} is the section area of the lateral force reinforcement per length s.

s is the distance of the lateral force reinforcement as measured along the component axis.

b_w is the web width of the component.

α is the angle between the lateral force reinforcement and the component axis.

The recommended value for the minimum reinforcement is:

$$\rho_{w,min} = 0.08 \sqrt{f_{ck}} / f_{yk} \quad (9.5N)$$

DIN EN 1992-1-1:

$$\rho_{w,min} = 0.16 f_{ctm} / f_{yk} \quad (9.5aDE)$$

With respect to slabs, the value can vary between zero and the above value as described in Chapter 9.3.2 (2).

For structured sections with prestressed tension chord, the following applies:

$$\rho_{w,min} = 0.256 f_{ctm} / f_{yk} \quad (9.5bDE)$$

OENORM B 1992-1-1:

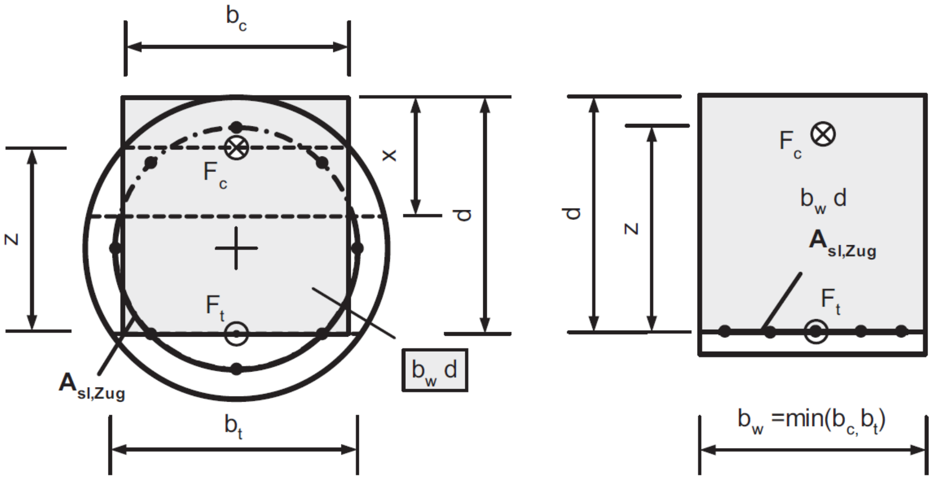
$$\rho_{w,min} = 0.15 f_{ctm} / f_{yd} \quad (24AT)$$

For slabs with a calculated shear reinforcement at least the 0.6-fold value of the minimum shear reinforcement of beams is to be applied.

Lateral force design for circular and annular cross-sections according to Bender et al.

For circular and annular cross sections with annular single stirrups or helixes, the lateral force design is optionally carried out according to Bender et al. (2010) for the resulting shear force $Q_r = \sqrt{(Q_y^2 + Q_z^2)}$.

In its interpretation of 1 June 2012 of Chapter 10.3 of DIN 1045-1:2008, the German Committee for Structural Engineering (NABau) recommends using the smaller value of the section width at the center of gravity of the steel tensile forces and the concrete compressive forces for the effective width b_w (see following figure). The values for the width b_w, the effective height d and the inner lever arm z are defined in the cross-section dialog.



Definition of the effective width bw as per NABau (2012) [Fig. from: Bender et al. (2006), p. 87]

For structural members without shear reinforcement, the resistance $V_{Rd,ct}$ is given according to Bender et al. (2006), Equ. (1), in accordance with DIN 1045-1:2008, Equ. (70). Therefore, the program uses the above equations (6.2a), (6.2b) and (11.6.2) of EN 1992-1-1 with the selected value for b_w.

For structural members with shear reinforcement, the design is carried out according to Bender et al. (2010):

$$V_{Rd,sy} = \alpha_k \cdot A_{sw} / s_w \cdot f_{yd} \cdot z \cdot \cot\Theta \cdot \sin\alpha \tag{10}$$

$$V_{Rd,max} = \alpha_k \cdot b_w \cdot z \cdot \alpha_c \cdot f_{cd} \cdot \cot\Theta / [(\cot\Theta \cdot \cot\alpha)^2 + 1] \tag{11}$$

where

α_k	is an efficacy factor, which is stress-dependent ($0.715 \leq \alpha_k \leq 0.785$) according to Bender et al. (2010), p. 422, and can be assumed with the mean value $\alpha_k = 0.75$.
A_{sw}	is the section area of the lateral force reinforcement per length s_w.
s_w	is the distance of the lateral force reinforcement as measured along the component axis.
b_w	is the effective cross-ection width.
z	is the inner lever arm.
Θ	is the inclination of the conrete compressive struts.
α	is the angle between the lateral force reinforcement and the component axis (helix inclination).
f_{yd}	is the design value for the yield strength of the lateral force reinforcement.
f_{cd}	is the design value of the concrete compressive strength.
α_c	is a coefficient to account for the stress state in the compression chord.

The additional tensile force in the longitudinal reinforcement due to lateral force Q_r is determined according to equation (6.18) of the standard. In case of simultaneous loading by lateral force and torsion, the torsion design is carried out according to the standard for vertical stirrups assuming a square torsion box.

The design results are stored separately from the standard design results.

Design for Torsion and Combined Stressing

The design for torsion is carried out according to EN 1992-1-1, Chapter 6.3. It includes the calculation of the diagonal tensile reinforcement and the longitudinal reinforcement based on Equation (6.28) and the concrete strut check under lateral force based on Formula (6.29) of the standard.

The equivalent section on which this design is based is defined by the user independently of the normal section geometry.

Strut angle

According to 6.3.2 (2), the rules set forth in Chapter 6.2.3 (2) for lateral force also apply for the strut angle.

DIN EN 1992-1-1:

For combined stress from torsion and proportional lateral force, V_{Ed} in Equation (6.7aDE) must include the shear force of the wall $V_{Ed,T+V}$ based on Equation (NA.6.27.1) and b_w in Equation (6.7bDE) must include the effective thickness of wall t_{ef}. The check for both lateral force and torsion must be carried out using the selected angle Θ. The reinforcements determined in this manner are to be added together.

$$V_{Ed,T+V} = V_{Ed,T} + V_{Ed} \cdot t_{ef} / b_w \tag{NA.6.27.1}$$

Alternatively a strut angle of 45° for torsion according to Chapter 6.3.2 (2) or a constant value $\cot\Theta$ for lateral force and torsion (cf. interpretation No. 24 of NABau for DIN 1045-1) can be chosen in the section dialog.

Torsion reinforcement

The necessary reinforcement is to be determined according to Chapter 6.3.2 (3):

$$\Sigma A_{sl} \cdot f_{yd} / u_k = T_{Ed} / 2A_k \cdot \cot\Theta \tag{6.28}$$

or

$$A_{sw} \cdot f_{yd} / s = T_{Ed} / 2A_k \cdot \tan\Theta$$

where

A_{sl}	is the section area of the longitudinal torsional reinforcement. The possibility of reducing the longitudinal torsional reinforcement in compression chords is not used.

A_{sw} is the section area of the torsion reinforcement perpendicular to the component axis.

u_k is the perimeter of area A_k.

s is the distance of the torsion reinforcement as measured along the component axis.

A_k is the area enclosed by the center lines of the walls.

For approximately rectangular full sections, only the minimum reinforcement defined in Section (5) is necessary if the condition expressed by Equation (6.31) is met:

$$T_{Ed} / T_{Rd,c} + V_{Ed} / V_{Rd,c} \leq 1.0 \quad (6.31)$$

where

$T_{Rd,c}$ is the torsion crack moment which, according to Zilch (2006, p. 290), is defined as $T_{Rd,c} = f_{ctd} \cdot W_T$.

$V_{Rd,c}$ is the lateral force resistance according to Equation (6.2).

DIN EN 1992-1-1:
The condition (6.31) is supplemented with the following equations:

$$T_{Ed} \leq \frac{V_{Ed} \cdot b_w}{4.5} \quad (NA.6.31.1)$$

$$V_{Ed}\left[1 + \frac{4.5\, T_{Ed}}{V_{Ed} \cdot b_w}\right] \leq V_{Rd,c} \quad (NA.6.31.2)$$

Strut load-bearing capacity

To avoid exceeding the strut load-bearing capacity of a component subject to torsion and lateral force, the following condition must be met:

$$T_{Ed} / T_{Rd,max} + V_{Ed} / V_{Rd,max} \leq 1.0 \quad (6.29)$$

where

T_{Ed} is the design value of the torsion moment.

V_{Ed} is the design value of the lateral force.

$T_{Rd,max}$ is the design value of the absorbable torsion moment based on

$$T_{Rd,max} = 2\,\nu \cdot \alpha_{cw} \cdot f_{cd} \cdot A_k \cdot t_{ef,i} \cdot \sin\Theta \cdot \cos\Theta \quad (6.30)$$

with α_{cw} according to Equation (6.9) and ν according to Eq. (6.6N) for normal concrete and according to Eq. (11.6.6N) for lightweight concrete.

DIN EN 1992-1-1:
For compact sections, the interaction Equation (NA.6.29.1) is used:

$$(T_{Ed} / T_{Rd,max})^2 + (V_{Ed} / V_{Rd,max})^2 \leq 1.0 \quad (NA.6.29.1)$$

In Equation (6.30) $\nu = \eta_1 \cdot 0.75$ is used for box sections and $\nu = \eta_1 \cdot 0.525 \cdot \min(1.0;\ 1.1 - f_{ck} / 500)$ in all other cases with $\eta_1 = 1$ for normal concrete and as per Eq. (11.1) for lightweight concrete.

OENORM B 1992-1-1:
For full sections the following interaction equation can be used:

$$(T_{Ed} / T_{Rd,max})^2 + (V_{Ed} / V_{Rd,max})^2 \leq 1.0 \quad (9AT)$$

Shear Joint Check

The shear joint check is peformed for beam elements and design objects in accordance with EN 1992-1-1, Chapter 6.2.5. It is carried out for the shear force in the z-direction of the cross-section and is only useful for components that are mainly stressed in this direction. The transmission of shear force through the joint is ensured if the following conditions are met:

$$v_{Edi} \leq v_{Rdi} \qquad (6.23)$$

Design value of the shear stress in the joint v_{Edi}:

$$v_{Edi} = \beta \cdot V_{Ed} \,/\, (z \cdot b_i) \qquad (6.24)$$

where

β is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered. The ratio factor β is calculated depending on whether the shear joint is in the compression or tension zone and whether reinforcement in the existing concrete or in the concrete supplement was determined from the other checks with the internal forces associated with V_{Ed} in state II as follows (see also Booklet 600 to 6.2.5 (1)):

Outer edge of concrete supplement	Shear joint	Tensile reinforcement		β
		Concrete supplement	Existing Concrete	
Tension	Ten. / Comp.	-	-	1
Compression	Compression	-	-	$0 < F_{cdi} / F_{cd} < 1$
	Tension	Yes	Yes	$0 < F_{sd} / (F_{sd} + F_{sdi}) < 1$
		Yes	No	0 (No check)
		No	Yes	1
		No	No	0 (No check)

z is the lever arm of the composite section. The lever arm is assumed as in the lateral force check according to the specifications in the shear section.

b_i is the width of the joint.

F_{cdi} is the concrete compression force in the concrete supplement in the compression zone.

F_{cd} is the total concrete compression force in the compression zone.

F_{sdi} is the tension force of the reinforcing steel layers in the concrete supplement in the tensile zone.

F_{sd} is the total tension force of the reinforcing steel layers in the tensile zone.

Design value of the shear resistance in the joint v_{Rdi}:

$$v_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_n + \rho \cdot f_{yd} \,(\mu \cdot \sin \alpha + \cos \alpha) \leq 0.5 \cdot \nu \cdot f_{cd} \qquad (6.25)$$

DIN EN 1992-1-1:

The load bearing component of the transverse reinforcement from the shear friction in Eq. (6.25) may be increased to $\rho \cdot f_{yd}\,(1.2 \cdot \mu \cdot \sin \alpha + \cos \alpha)$.

For very smooth joints with external compression normal force perpendicular to the joint, the friction component in Eq. (6.25) may be taken into account up to the limit $\mu \cdot \sigma_n \leq 0.1 \cdot f_{cd}$ according to NPD 6.2.2 (6).

where

c, μ are factors which depend on the roughness of the joint (see 6.2.5 (2)). Under dynamic or fatigue loads, the value c is halved according to 6.2.5 (5).

DIN EN 1992-1-1:

For very smooth joints under dynamic or fatigue loads, c = 0 is to be assumed.

f_{ctd} is the design value of the concrete tensile strength according to 3.1.6 (2)P.

σ_n is the stress caused by the minimum normal force perpendicular to the joint, which can act simultaneously with the lateral force (positive for compression with $\sigma_n < 0.6 f_{cd}$ and negative for tension). If σ_n is a tensile stress $c \cdot f_{ctd}$ should usually be set to 0.

$\rho = A_s / A_i$

A_s is the area of reinforcement crossing the joint, including ordinary shear reinforcement, with adequate anchorage at both sides of the interface.

A_i is the area of the joint.

α the angle of inclination of the transverse reinforcement. This is set at 90° by the program.

v is a strength reduction factor according to 6.2.2 (6).

OENORM B 1992-1-1:
If reinforcement is required, the following minimum reinforcement must be inserted perpendicular to the joint for beam-like components:

$\rho_{min} = 0.2 \cdot f_{ctm} / f_{yk} \geq 0.001$

Punching Shear

The load-bearing safety check against punching shear is carried out according to EN 1992-1-1, Chapter 6.4. This check is used to determine the necessary punching reinforcement. The following special conditions apply:

- The orthogonal effect directions labeled as y and z in the standard are indicated below as well as in the dialog and the listing as x and y in order to comply with the area reinforcement directions commonly used in the program.
- The average effective static height d results from the input parameters d_x and d_y with $d = (d_x + d_y) / 2$. These parameters are to be selected as shown in Figure 6.12, 6.16 or 6.17.
- No check is carried out for pad footings.
- The action can be entered directly or taken from the analyzed design situation at the ultimate limit state. In this case, V_{Ed} is set to the maximum support force R_z for each corresponding action combination.
- BS EN 1992-1-1:
  The shear strength of concrete of strength classes higher than C50/60 is limited to the value of class C50/60, according to NA to 3.1.2 (2)P.

The check is considered fulfilled if:

1. For slabs w i t h o u t punching reinforcement

   $v_{Ed} \leq v_{Rd,max}$,

   $v_{Ed} \leq v_{Rd,c}$.

2. For slabs w i t h punching reinforcement

   $v_{Ed} \leq v_{Rd,max}$,

   $v_{Ed} \leq v_{Rd,cs}$.

DIN EN 1992-1-1, OENORM B 1992-1-1:
3. The minimum longitudinal reinforcement is maintained

with

$$v_{Ed} = \frac{\beta \cdot V_{Ed}}{u_i \cdot d} \tag{6.38}$$

where

v_{Ed} is the maximum acting lateral force per area unit.

V_{Ed} is the design value of the entire lateral force to be absorbed. For foundation slabs the lateral force may be reduced due to the favorable action of the soil pressure according to 6.4.3 (8). For the reduction the program assumes the area within u_1 unless the national annex contains a different rule.

β is the load increase factor for taking into account eccentric load introduction according to Equation (6.39). The value is specified by the user.

DIN EN 1992-1-1, OENORM B 1992-1-1:
Values smaller then 1.10 are not permitted.

d is the average effective height of the slab, which can be assumed as $(d_x + d_y) / 2$, with:

d_x, d_y is the effective static height of the slab in the x or y direction in the section area of the analyzed perimeter.

u_i is the circumference of the analyzed perimeter.

$v_{Rd,c}$ is the design value of the punching resistance per area unit for a slab without punching reinforcement.

$v_{Rd,cs}$ is the design value of the punching resistance per area unit for a slab with punching reinforcement.

$v_{Rd,max}$ is the design value of the maximum punching resistance per area unit.

The load discharge areas and check sections as per Chapter 6.4.2, Section (1) to (7), are taken into consideration. The user-specified opening dimensions are used to calculate the check sections.

Punching resistance without punching reinforcement

The punching resistance of a slab without punching reinforcement is calculated as

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100\ \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp} \geq (\ v_{min} + k_1 \cdot \sigma_{cp}\ ) \quad (6.47)$$

For lightweight concrete applies

$$v_{lRd,c} = C_{lRd,c} \cdot k \cdot \eta_1 \cdot (100\ \rho_l \cdot f_{lck})^{1/3} + k_2 \cdot \sigma_{cp} \geq (\ \eta_1 \cdot v_{l,min} + k_2 \cdot \sigma_{cp}\ ) \quad (11.6.47)$$

where

f_{ck} is the characteristic concrete strength [N/mm²]

k $= 1 + \sqrt{(200 / d)} \leq 2.0$; d [mm]

ρ_l $= \sqrt{(\rho_{lx} \cdot \rho_{ly})} \leq 0.02$

ρ_{lx}, ρ_{ly} is the reinforcement level based on the fixed tensile reinforcement in the x or y direction. The values ρ_{lx} and ρ_{ly} are normally calculated as average values given a slab width based on the column measurements plus $3 \cdot d$ for each side.

σ_{cp} $= (\sigma_{cx} + \sigma_{cy}) / 2$

where

σ_{cx}, σ_{cy} are the normal stresses in the concrete in the x and y directions in the critical section (MN/m², positive for pressure):
$\sigma_{cx} = N_{Ed,x} / A_{cx}$ and $\sigma_{cy} = N_{Ed,y} / A_{cy}$.

$N_{Ed,x}$, $N_{Ed,y}$ are the normal forces acting on internal columns in the entire section area of the analyzed perimeter and the normal forces acting on the edge and corner columns in the area of the analyzed perimeter. These forces result from loads or prestressing.

A_c is the section area of the concrete according to the definition of N_{Ed}.

The recommended country-specific values are:

$C_{Rd,c}$ $= 0.18 / \gamma_c$ for normal concrete

$C_{lRd,c}$ $= 0.15 / \gamma_c$ for lightweight concrete

k_1 $= 0.1$

k_2 $= 0.08$

v_{min} derived from Equation (6.3N) for normal concrete:

$$v_{min} = 0.035 \cdot k^{3/2} \cdot f_{ck}^{1/2} \quad (6.3N)$$

$v_{l,min}$ derived from Chapter 11.6.1 (1) for lightweight concrete:

$$v_{l,min} = 0.028 \cdot k^{3/2} \cdot f_{lck}^{1/2}$$

For column foundations and foundation slabs the following applies according to Chapter 6.4.4 (2):

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100\ \rho_l \cdot f_{ck})^{1/3} \cdot 2 \cdot d/a \geq (\ v_{min} \cdot 2 \cdot d/a\ ) \quad (6.50)$$

$$v_{lRd,c} = C_{lRd,c} \cdot k \cdot \eta_1 \cdot (100\ \rho_l \cdot f_{lck})^{1/3} \cdot 2 \cdot d/a \geq (\ \eta_1 \cdot v_{l,min} \cdot 2 \cdot d/a\ ) \quad (11.6.50)$$

a Distance from the column edge to the decisive perimeter. Within the perimeter the soil pressures deducting the foundation dead load are allowed for relieving.

DIN EN 1992-1-1:

v_{min} as in Section 6.2.2 (1)

For internal columns of flat slabs with $u_0/d < 4$ the following applies according to Book 600 of the DAfStb:

$C_{Rd,c} = C_{lRd,c} = 0.18 / \gamma_c \cdot (0.1 \cdot u_0 /d + 0.6) \geq 0.15/\gamma_c$ (H.6-16)

For column foundations and foundation slabs the following applies:

a The program does not iterate over the distance a according to NCI of 6.4.4 (2). If a constant perimeter with $a = 1.0\ d$ is assumed, only 50% of the soil pressures are allowed for relieving. This is taken into account during determination of the resistance.

$C_{Rd,c} = C_{lRd,c} = 0.15 / \gamma_c$

In all other cases the recommended value for $C_{Rd,c}$ applies.

The bending reinforcement level ρ_l must also be limited to $\rho_l \leq 0.5 f_{cd} / f_{yd}$.

OENORM B 1992-1-1:

The allowable reinforcement level for determining $v_{Rd,c}$ must not exceed

$\rho_l = 0.4 \cdot f_{cd} / f_{yd} \leq 0.02$.

For foundation slabs the program does not iterate over the distance a according to the supplement to 6.4.4 (2). For simplification, a constant perimeter with $a = 1.0\ d$ can be assumed.

Punching resistances with punching reinforcement

1) The punching resistances with punching reinforcement are calculated as

$v_{Ed} = \beta \cdot V_{Ed} / (u_0 \cdot d) \leq v_{Rd,max}$ for normal concrete (6.53)

$v_{Ed} = V_{Ed} / (u_0 \cdot d) \leq v_{lRd,max}$ for lightweight concrete (11.6.53)

where

u_0 For an internal column u_0 = Circumference of the column

For an edge column $u_0 = c_2 + 3 \cdot d \leq c_2 + 2 \cdot c_1$

For a corner column $u_0 = 3 \cdot d \leq c_1 + c_2$

c_1, c_2 are the column dimensions as shown in Figure 6.20. For circular columns, the values c_1, c_2 in the equations for edge and corner columns are assumed to be $c_1 = c_2 = 0.25 \cdot$ column circumference. For wall ends and wall corners, u_0 is determined corresponding to edge and corner columns. Defined openings are taken into account by reducing u_0 as for the critical perimeter.

OENORM B 1992-1-1: The possibility of simplification in case of round edge and corner columns is not used.

β See 6.4.3 (3), (4) and (5).

v_{Ed} is the lateral force to be absorbed at the column section per area unit.

The recommended country-specific values are:

$v_{Rd,max} = 0.4 \cdot \nu \cdot f_{cd}$ with ν as per Eq. (6.6N)

$v_{lRd,max} = 0.4 \cdot \nu \cdot f_{lcd}$ with ν as per Eq. (11.6.6N)

DIN EN 1992-1-1:

The maximum punching resistance is determined within the critical perimeter u_1 :

$v_{Ed,u1} \leq v_{Rd,max} = 1.4 \cdot v_{Rdc,u1}$ (NA.6.53.1)

Deviating from NDP of 6.4.5 (3), $v_{Rdc,u1}$ is set to $v_{Rd,c}$ according to Eq. (6.47).

OENORM B 1992-1-1:

For slabs or foundations with small load introduction areas with $u_0/d < 4$, the maximum punching resistance $v_{Rd,max}$ shall be limited to the recommended values.

SS EN 1992-1-1:

$v_{Rd,max} \leq 0.50 \cdot \nu \cdot f_{cd}$ (Article 16)

$v_{Rd,max} \leq 0.50 \cdot \nu \cdot f_{lcd}$ (Article 36a)

BS EN 1992-1-1:

$v_{Rd,max} = 0.5 \cdot \nu \cdot f_{cd}$

For lightweight concrete the recommended value applies. The concrete compressive strength f_{cd} according to Eq. (3.15) is determined conservatively with $\alpha_{cc} = 0.85$.

2) The first reinforcement row is specified with a distance of $0.5 \cdot d$ from the column edge; the other reinforcement rows are specified with a distance of $s_r \leq 0.75 \cdot d$ from each other (see Figure 9.10). The reinforcement is determined using the following equation:

$$v_{Rd,cs} = 0.75 \cdot v_{Rd,c} + 1.5 \cdot (d / s_r)\, A_{sw} \cdot f_{ywd,ef} \;\; (1/(u_1 \cdot d)) \sin \alpha \leq k_{max} \cdot v_{Rd,c} \;\; [\text{MN/m}^2] \tag{6.52}$$

where

A_{sw}	is the section area of the punching reinforcement in a reinforcement row around the column [mm²].
s_r	is the radial distance of the punching reinforcement rows [mm].
$f_{ywd,ef}$	is the effective design value for the yield strength of the punching reinforcement according to $f_{ywd,ef} = 250 + 0.25 \cdot \mathrm{d} \leq f_{ywd}$ [MN/m²]
u_1	is the circumference of the critical perimeter [mm].
d	is the average value of the effective static heights in the orthogonal directions [mm].
α	is the angle between the punching reinforcement and the slab plane.
k_{max}	is the factor for limiting the maximal load-bearing capacity with punching reinforcement. The recommended value is 1.5.

BS EN 1992-1-1: k_{max} = 2.0

SS EN 1992-1-1: k_{max} = 1.6

For bent-down reinforcement $d / s_r = 0.67$ is used according to Section 6.4.5 (1).

DIN EN 1992-1-1:

Within the perimeter u_{out} as per Section 6.4.5 (4), a minimum of two reinforcement rows is always required. For the first two reinforcement rows of flat slabs, A_{sw} is to be increased by the factors $k_{sw,1} = 2.5$ resp. $k_{sw,2} = 1.4$. For bent-down punching shear reinforcement $d / s_r = 0.53$ is used. The bent-down reinforcement can be exploited with $f_{ywd,ef} = f_{ywd}$. For foundation slabs the stirrup reinforcement is calculated according to the following Equation:

$$\beta \cdot V_{Ed,red} \leq V_{Rd,s} = A_{sw,1+2} \cdot f_{ywd,ef} \tag{NA.6.52.1}$$

For bent-down reinforcement:

$$\beta \cdot V_{Ed,red} \leq V_{Rd,s} = 1.3\, A_{sw,1+2} \cdot f_{ywd} \sin \alpha \tag{NA.6.52.2}$$

In order to determine $V_{Ed,red}$, the reduction value ΔV_{Ed} in Eq. (6.48) is calculated using the area A_{crit} according to Figure NA.6.12.1 for the first two reinforcement rows and $A_i > A_{crit}$ for the following rows $i > 2$, whereas in each case only 50% of the soil pressures are applied relieving. For the first two rows, 50% of the reinforcement amount determined with Eq. (NA.6.52.1) are required whereas 33% should be installed in each of the following rows. The first reinforcement row is specified with a distance of $0.3 \cdot d$ from the column edge. For the first three rows the distance s_r between the rows should be limited to $0.5 \cdot d$.

OENORM B 1992-1-1:

For each of the first two rows A_{sw} is to be increased by 60%. The coefficient k_{max} in Eq. (6.52) is assumed to be k_{max} = 1.65. According to OENORM, it is assumed that the punching reinforcement comprises the respective lower layer of the bending reinforcement or consists of bent-up bars. For the given normal force N_{Ed}, it is assumed that it does not include a compressive normal force from prestressing.

3) The perimeter $u_{out,ef}$, which does not need any punching reinforcement, is normally calculated based on Equation (6.54):

$$u_{out,ef} = \beta \cdot V_{Ed} / (v_{Rd,c} \cdot d) \tag{6.54}$$

In general, the outermost row of the punching reinforcement must not be farther from $u_{out,ef}$ than $1.5 \cdot d$.

DIN EN 1992-1-1:

$v_{Rd,c}$ is determined as the lateral force resistance according to Chapter 6.2.2 (1).

4) For the minimum required punching reinforcement $A_{sw,min}$ of the internal check sections, the following applies:

$$A_{sw,min} \cdot \left(1.5 \cdot \sin\alpha + \cos\alpha\right) / \left(s_r \cdot s_t\right) \geq 0.08 \cdot \sqrt{\left(f_{ck}\right)} / f_{yk} \qquad (9.11)$$

where

α is the angle between the punching reinforcement and the longitudinal reinforcement (i.e., for vertical stirrups α = 90° and sin α = 1).

s_r is the radial distance of the stirrups of the punching reinforcement.

s_t is the tangential distance of the stirrups of the punching reinforcement.

f_{ck} in N/mm².

DIN EN 1992-1-1:

$$A_{sw,min} = A_s \cdot \sin\alpha = 0.08 / 1.5 \cdot \sqrt{f_{ck}} / f_{yk} \cdot s_r \cdot s_t \qquad (9.11DE)$$

5) Minimum longitudinal reinforcement

DIN EN 1992-1-1:

The minimum reinforcement is found according to Chapter 6.4.5 (NA.6) based on the design of the minimum moments:

$m_{Ed,x} = \eta_x \cdot V_{Ed}$ and $m_{Ed,y} = \eta_y \cdot V_{Ed}$ (NA.6.54.1)

with

η_x, η_y the moment coefficient as per Table NA.6.1.1 for the x or y direction.

OENORM EN 1992-1-1:

The minimum reinforcement is determined according to Equation (28AT):

$$a_{s,min} = \frac{V_{Ed}}{0.9 \cdot d \cdot f_{yd}} \cdot \frac{e}{b_{eff}} \qquad (28AT)$$

with

e/b_{eff} the relative eccentricity as per Table 14AT.

Checks Against Fatigue

The following checks according to EN 1992-1-1 are available:

- Fatigue of longitudinal reinforcement, shear reinforcement and prestressing steel (Chapter 6.8.5, 6.8.6)
- Fatigue of concrete under compressive stress (Chapter 6.8.7)
- Fatigue of the concrete compressive struts under lateral force and torsion (Chapter 6.8.7 (3))

The user can select two alternative methods for design:

- Simplified check for the frequent action combination according to EN 1992-1-1, Chapter 6.8.6 (2), and EN 1990, Eq. (6.15b), taking the relevant traffic loads at serviceability limit state into account.
- Check with damage equivalent stress ranges for the fatigue combination according to EN 1992-1-1, Chapter 6.8.3, Eq. (6.69), considering the specific fatigue load Q_{fat}.

The concrete compressive stresses are determined for both cases in state II. Differing from Chapter 5.10.9 the variation of prestressing is not taken into account.

Design Combinations

For the check against fatigue two alternative action combinations can be used:

- Frequent combination for simplified checks according to EN 1992-1-1, Chapter 6.8.6 (2) in conjunction with EN 1990, Chapter 6.5.3.

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} \psi_{1,1} \cdot Q_{k,1} \text{"+"} \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \qquad (6.15b)$$

- Fatigue combination for checks with damage equivalent stress ranges.

$$\left(\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} \psi_{1,1} \cdot Q_{k,1} \text{"+"} \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \right) \text{"+"} Q_{fat} \qquad (6.69)$$

In this equation $Q_{\mathrm{k,1}}$ and $Q_{\mathrm{k,i}}$ are non-cyclic, non-permanent actions whereas Q_{fat} defines the relevant fatigue load.

For each combination you can define different design situations for the construction stages and final states. When conducting the check, the extreme value deriving from all combinations and situations is decisive.

Stress-Strain Curves

For checks against fatigue the following characteristics apply:

- Concrete: Stress-strain curve according to EN 1992-1-1, Figure 3.2, where a horizontal curve is assumed for strains of ε_{c1} or higher (cf. Rossner, Graubner 2012, p. 511 and Nguyen et al. 2015, p. 158).
- Reinforcing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.8, with rising upper branch, where the maximum stress is assumed to be $k \cdot f_{\mathrm{yk}}$ with $k = 1.05$ as per Table C.1, class A.

  DIN EN 1992-1-1:

  The maximum stress is assumed to be 1.05 $\cdot f_{\mathrm{yk}} / \gamma_s$ for ductility class A according to DIN 488-1.
- Prestressing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.10, with horizontal upper branch according to Chapter 3.3.6 (7) of the standard and a maximum stress of $f_{\mathrm{p;0,1k}}$.

Fatigue of Longitudinal Reinforcement, Shear Reinforcement and Prestressing Steel

The fatigue check is carried out according to EN 1992-1-1, Chapter 6.8. The steel stresses are calculated for longitudinal reinforcement from bending and longitudinal force as well as for prestressing steel in beams and design objects under the assumption of a cracked concrete section. For shear and longitudinal reinforcement from lateral force and torsion, the stresses are calculated according to 6.8.2 (3) based on a truss model with the strut angle $\tan \Theta_{fat} = \sqrt{\tan \Theta} \leq 1$ acc. to Eq. (6.65). Where Θ is the angle between the concrete compression struts and the beam axis used in the corresponding ultimate limit state design. The prestressing steel stresses in area elements are determined at the uncracked concrete section. Tendons without bond and external tendons are not checked.

DIN EN 1992-1-1:

The strut angle is to be determined according to Eq. (H.6-26) of Book 600 of the DAfStb.

Simplified check

According to Chapter 6.8.6, adequate fatigue resistance may be assumed if the stress range under the frequent action combination does not exceed 70 MN/m² for unwelded reinforcing bars and 35 MN/m² for welded bars. The condition described in Chapter 6.8.6 (3) for couplings in prestressed components is not examined by the program.

DIN EN 1992-1-1:

The simplified check is not permitted for welded reinforcing bars.

Check with damage equivalent stress ranges

According to Chapter 6.8.5 (3), for reinforcing and prestressing steel adequate fatigue resistance should be assumed if Eq. (6.71) is satisfied:

$$\gamma_{F,fat} \cdot \Delta\sigma_{s,equ}(N^*) \leq \Delta\sigma_{Rsk}(N^*) / \gamma_{s,fat} \qquad (6.71)$$

with

$\gamma_{F,fat}$	= 1 according to Chapter 2.4.2.3 and 6.8.4 (1).
$\gamma_{s,fat}$	= 1.15 for reinforcing and prestressing steel according to Chapter 2.4.2.4.
$\Delta\sigma_{Rsk}(N^*)$	Permitted characteristic stress range at N^* load cycles based on the S-N curves specified in Tab. 6.4N for prestressing steel or Tab. 6.3N for reinforcing steel.
$\Delta\sigma_{s,equ}(N^*)$	Damage equivalent stress range with N^* load cycles. For building construction this value may be approximated by $\Delta\sigma_{s,max}$.
$\Delta\sigma_{s,max}$	Maximum stress range from the fatigue combination.

Calculation method

The maximum from the robustness, crack and bending reinforcement is taken as the existing bending reinforcement. If as a result the load from the fatigue combination in state II cannot be absorbed, the design will be repeated using the existing reinforcement and the check internal forces.

The maximum stress range per steel layer that results from the strain state in state II or the truss model is determined separately for each check situation. For longitudinal reinforcement the varying bond behavior of reinforcing and prestressing steel is taken into account by increasing the steel stress by the coefficient η from Eq. (6.64). If for longitudinal and shear reinforcement the resulting stress range exceeds the permitted stress range, the necessary reinforcement will be iteratively increased until the check succeeds for all situations. In the *Symmetrical* and *Compression member* design modes the longitudinal reinforcement is applied at all predefined locations. This will not affect the predefined relationships between the individual reinforcement layers.

The permitted stress ranges and the coefficient η are specified by the user in the Section dialog.

The decisive reinforcement used for the check, which may have been increased, is recorded in the check log and saved for graphical representation.

Fatigue of Concrete Under Longitudinal Compressive Stress

The fatigue check for concrete that is subject to compressive stress is performed for bending and longitudinal force at the cracked section as described in EN 1992-1-1, Chapter 6.8.7. This check takes into account the final longitudinal reinforcement and may include an increase applied during the fatigue check for reinforcing steel.

Simplified check

Adequate fatigue resistance may be assumed if the following condition is satisfied:

$$\frac{\sigma_{c,\max}}{f_{cd,fat}} \leq 0.5 + 0.45 \frac{\sigma_{c,\min}}{f_{cd,fat}} \begin{array}{l} \leq 0.9 \text{ for } f_{\mathrm{ck}} \leq 50 \text{ MN/m}^2 \\ \leq 0.8 \text{ for } f_{\mathrm{ck}} > 50 \text{ MN/m}^2 \end{array} \qquad (6.77)$$

where

$\sigma_{\mathrm{c,max}}$ is the maximum compressive stress at a fibre under the frequent load combination (compression measured positive).

$\sigma_{\mathrm{c,min}}$ is the minimum compressive stress at the same fibre where $\sigma_{\mathrm{c,max}}$ occurs ($\sigma_{\mathrm{c,min}} = 0$ if $\sigma_{\mathrm{c,min}}$ is a tensile stress).

$f_{\mathrm{cd,fat}}$ is the design fatigue strength of concrete according to Eq. (6.76). This value is entered by the user in the Section dialog.

$f_{\mathrm{cd,fat}}$ $= k_1 \cdot \beta_{\mathrm{cc}}(t_0) \cdot f_{\mathrm{cd}} \cdot (1 - f_{\mathrm{ck}} / 250)$ (6.76)
with $\beta_{\mathrm{cc}}(t_0)$ as per Eq. (3.2) and f_{cd} as per Eq. (3.15)

k_1 $= 0.85$

DIN EN 1992-1-1, OENORM B 1992-1-1, SS EN 1992-1-1:

k_1 $= 1.0$

Check with damage equivalent concrete compressive stresses

According to Chapter 6.8.7 (1), a satisfactory fatigue resistance may be assumed for concrete under compression, if Eq. (6.72) is fulfilled:

$$E_{cd,\max,equ} + 0.43\sqrt{1 - R_{equ}} \leq 1 \qquad (6.72)$$

where

R_{equ} $= E_{\mathrm{cd,min,equ}} / E_{\mathrm{cd,max,equ}}$ is the stress ratio.

$E_{\mathrm{cd,min,equ}}$ $= \sigma_{\mathrm{cd,min,equ}} / f_{\mathrm{cd,fat}}$ is the minimum compressive stress level.

$E_{\mathrm{cd,max,equ}}$ $= \sigma_{\mathrm{cd,max,equ}} / f_{\mathrm{cd,fat}}$ is the maximum compressive stress level.

$\sigma_{\mathrm{cd,min,equ}}$ is the lower stress of the ultimate amplitude for $N = 10^6$ cycles.

$\sigma_{\mathrm{cd,max,equ}}$ is the uper stress of the ultimate amplitude for $N = 10^6$ cycles.

$f_{\mathrm{cd,fat}}$ is the design fatigue strength of concrete according to Eq. (6.76).

Fatigue of the Concrete Compressive Struts Under Lateral Force and Torsion

Fatigue of the concrete compressive struts is examined for beams and design objects. The check differentiates between components with and without calculatory required lateral force. In the case of combined loads from lateral force and torsion, the supplementary condition according to Chapter EN 1992-1-1, 6.3.2 (5) is checked in addition to the condition in Chapter 6.2.1 (5).

DIN EN 1992-1-1: In addition, the equations according to NCI for 6.3.2 (5) are evaluated.

Components with required lateral force reinforcement

The fatigue check for concrete under compressive stress as per Chapter 6.8.7, is also applicable for verifying the concrete compressive struts of components with required lateral force reinforcement as per Chapter 6.8.7 (3).

In the case of vertical stirrups ($\alpha = 90°$), the design values $\sigma_{cd,max}$ and $\sigma_{cd,min}$ of the maximal and minimal compressive stress may be determined according to the following equations while assuming an identical compressive strut angle θ for lateral force and torsion as well:

$$\sigma_{cd,T} = \frac{T_{Ed}}{2 \cdot A_k \cdot t_{ef}} \cdot (\cot\Theta + \tan\Theta)$$

$$\sigma_{cd,V} = \frac{V_{Ed}}{b_w \cdot z} \cdot (\cot\Theta + \tan\Theta)$$

$$\sigma_{cd,max} = \begin{cases} \max \sigma_{cd,T} + \text{cor. } \sigma_{cd,V} \\ \max \sigma_{cd,V} + \text{cor. } \sigma_{cd,T} \end{cases}$$

$$\sigma_{cd,min} = \begin{cases} \min \sigma_{cd,T} + \text{cor. } \sigma_{cd,V} \\ \min \sigma_{cd,V} + \text{cor. } \sigma_{cd,T} \end{cases}$$

The program performs the check depending on the user's selection either according to the simplified method as per Eq. (6.77), for the frequent combination or by using the damage equivalent stress range as per Eq. (6.72), for the fatigue combination given in Chapter 6.8.3, Eq. (6.69).
When performing the simplified check under pure lateral force load, the concrete strength $f_{cd,fat}$ should be reduced by the factor $\nu \cdot \eta_1$ as per Chapter 6.2.2 (6). In case of combined stressing from lateral force and torsion, the reduction factor $\nu \cdot \alpha_{cw} \cdot \eta_1$ with α_{cw} as per Eq. (6.9) applies. The coefficient η_1 should be set to 1 for normal concrete and according to Eq. (11.1), for light weight concrete.

DIN EN 1992-1-1:
The following factors apply:

$\nu_1 = 0.75 \cdot \nu_2 \cdot \eta_1$	in case of pure lateral force as per NDP 6.2.3 (3).
$\nu_2 = (1.1 - f_{ck} / 500) \leq 1.0$	acc. to NCI 6.8.7 (3).
$\nu \cdot \alpha_{cw} \cdot \eta_1 = 0.525 \cdot \eta_1$	in case of combined stressing as per NDP 6.2.2 (6).

Components without required lateral force reinforcement

For components without required lateral force reinforcement at ultimate limit state, adequate fatigue resistance against lateral force load may be assumed according to Chapter 6.8.7 (4), if the following conditions are satisfied:

$$\text{for } \frac{V_{Ed,min}}{V_{Ed,max}} \geq 0 : \frac{|V_{Ed,max}|}{|V_{Rd,c}|} \leq 0.5 + 0.45 \cdot \frac{|V_{Ed,min}|}{|V_{Rd,c}|} \begin{cases} \leq 0.9 \text{ for concrete up to } C50/60 \\ \leq 0.8 \text{ for concrete } C55/67 \text{ or higher} \end{cases} \tag{6.78}$$

$$\text{for } \frac{V_{Ed,min}}{V_{Ed,max}} < 0 : \frac{|V_{Ed,max}|}{|V_{Rd,c}|} \leq 0.5 - \frac{|V_{Ed,min}|}{|V_{Rd,c}|} \tag{6.79}$$

where

$V_{Ed,max}$	is the design value of the maximum lateral force under the frequent action combination.
$V_{Ed,min}$	is the design value of the minimum lateral force under the frequent action combination at the cross-section where $V_{Ed,max}$ occurs.
$V_{Rd,c}$	is the design value of the absorbable lateral force without shear reinforcement as per Eq. (6.2a).

For performing the check, the program selects automatically the simplified method with the frequent action combination.

Special Characteristic of Shell Structures

In shell structures the strain state at the cracked concrete section under general stress cannot be determined unambiguously. The design is therefore carried out separately for the reinforcement directions x and y with the design internal forces from Wolfensberger/Thürlimann or Rüsch as described above. The reinforcement calculated in this manner yields a reliable load-bearing capacity.

When calculating the stress range for reinforcing steel and concrete, this method can lead to unrealistic results in the case of torsional or shear stresses as shown in the following example:

Assume two identical sets of slab internal forces:

Set	mx [kNm/m]	my [kNm/m]	mxy [kNm/m]
1	300	200	100
2	300	200	100

According to Wolfensberger/Thürlimann, this results in design variants for the x direction:

Set	Variant	m [kNm/m]
1	1	mx + |mxy| = 400
	2	mx – |mxy| = 200
2	1	mx + |mxy| = 400
	2	mx – |mxy| = 200

The torsional moments generate a variation of the design moments and thus a calculatory stress range. This may lead to a necessary reinforcement increase in the fatigue check due to apparent overstressing. For normal design forces, this applies correspondingly to the shear forces.

Selecting ***Limit design variants*** in the Section dialog allows you to avoid the described effect. In this case only the corresponding variants are compared when determining the stress range, i.e. only the first and second variants of both sets in this example. Assuming constant stress, the stress range is thus correctly determined to be zero.

This alternative, however, does not ensure that all conceivable stress fluctuations are analyzed. You should therefore be particularly careful when assessing the results. For this purpose the detailed log indicates the main variants and design internal forces used for the check.

When determining the design internal forces according to Rüsch for inclined reinforcement, the described relationships apply accordingly.

Checks in the Serviceability Limit States

The following checks are performed:

- Limiting the concrete compressive stresses (EN 1992-1-1, Chapter 7.2).
- Limiting the reinforcing steel stresses (Chapter 7.2).
- Limiting the prestressing steel stresses (Chapter 7.2).
- Decompression check (Chapter 7.3.1).
- Minimum reinforcement for crack width limitation (Chapter 7.3.2).
- Crack with calculation (Chapter 7.3.4).
- Crack width by limitation of the bar distances (Chapter 7.3.3 (2)).
- Limiting deformations (Chapter 7.4).

Design Combinations

In accordance with EN 1990 (Eurocode 0), Chapter 6.5.3, the following combinations are taken into account in the serviceability limit states:

- Combination for characteristic situations

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{0,i} \cdot Q_{k,i} \qquad (6.14b)$$

- Combination for frequent situations

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \psi_{1,1} \cdot Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \qquad (6.15b)$$

- Combination for quasi-continuous situations

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \qquad (6.16b)$$

For each combination you can define different design situations for the construction stages and final states. If necessary, the combination required by the check will automatically be determined from the section specifications. Each check is carried out for all the situations of a combination.

Stress-Strain Curves

For checks in the serviceability limit states the following characteristics apply:

- Concrete: Stress-strain curve according to EN 1992-1-1, Figure 3.2, where a horizontal curve is assumed for strains of ε_{c1} or higher (cf. Interpretation No. 098 of the NABau for DIN TR 102).
- Reinforcing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.8, with rising upper branch, where the maximum stress is assumed to be $k \cdot f_{yk}$ with $k = 1.05$ as per Table C.1, class A.

  DIN EN 1992-1-1:
  The maximum stress is assumed to be $1.05 \cdot f_{yk}$ for ductility class A according to DIN 488-1.
- Prestressing steel: Stress-strain curve according to EN 1992-1-1, Figure 3.10, with horizontal upper branch according to Chapter 3.3.6 (7) of the standard and a maximum stress of $f_{p;0,1k}$.

Stress Analysis

For uncracked concrete sections, the program assumes that concrete and steel under tensile and compressive stress behave elastically. As for cracked concrete sections, the concrete compressive stresses are determined using the aforementioned stress-strain curve.

Area elements

For area elements the concrete stresses are calculated at the gross section. The steel stress check is carried out for reinforcing steel by determining the strain state at the cracked concrete section and for the prestressing steel at the uncracked concrete section.

Beams and design objects

The action combination stresses that can be determined without checks are always calculated at the gross section.

Conversely, in the checks the stresses are determined as follows and are graphically displayed or logged:

- When checking the crack reinforcement and crack width, the concrete stress is calculated at the gross section
- When checking the decompression and concrete compressive stresses, the concrete stress is calculated
  - without internal tendons at the gross section
  - with internal tendons without bond at the net section
  - with internal tendons with bond for situations before being grouted at the net section or otherwise at the ideal section
- The reinforcing and prestressing steel stresses are checked by determining the strain state at the cracked concrete section

OENORM B 1992-1-1:

If the stresses according to Chapter 7.2 are calculated at the cracked section the different bonding behavior of concrete and prestressing steel is to be taken into account. The increase of tension force ΔF_{tp} in the prestressing steel is to be calculated as follows:

$$\Delta F_{tp} = \xi_1^2 \cdot A_p \cdot \varepsilon(y_p) \cdot E_p \tag{13AT}$$

where

ξ_1 is the bond coefficient according to Eq. (7.5); the value can be entered in the crack width check dialog;

A_p is the section area of the tendon with bond;

$\varepsilon(y_p)$ is the strain of the concrete section at the location y_p of the tendon;

E_p is the elasticity modulus of the tendon.

For beams and design objects this rule is taken into account by the program for situations after grouting. For area elements it is not used because prestressing steel is only checked at the uncracked section.

Limiting the Concrete Compressive Stresses

The concrete compressive stress check is carried out according to EN 1992-1-1, Chapter 7.2. As described in Chapter 7.1 (2), a cracked section is assumed if the tensile stress calculated in the uncracked state exceeds f_{ctm}.

The calculation in the cracked state is performed by determining the strain state with the final longitudinal reinforcement (maximum from robustness, crack and bending reinforcement including a possible increase from the fatigue check). For beams and design objects, the tendons with bond are taken into account on the resistance side provided that they are grouted in the check situation. For area elements, the compressive stress for both reinforcement directions is determined separately and the extreme value is checked because the general strain state cannot be determined unambiguously.

In the construction stages and final states, for members of exposure classes *XD*, *XF* and *XS* the concrete compressive stress σ_c as defined in Chapter 7.2 (1) is to be limited to $0.60 f_{ck}$ under the characteristic combination. If stress in the concrete under quasi-continuous combination does not exceed the limit $0.45 f_{ck}$, linear creep can be assumed according to 7.2 (3). If this is not the case, non-linear creep must be taken into account. Both conditions are considered based on the user's specifications.

In prestressed concrete components as per Chapter 5.10.2.2, the maximum concrete compressive stress must be limited to $0.60 f_{c(t)}$ when entering the average prestressing value. If the concrete compressive stress exceeds the value $0.45 f_{c(t)}$, the nonlinearity of the creep must be taken into account. $f_{c(t)}$ indicates the average value of the concrete compressive strength at time t when the prestressing is entered.

The program assumes the time of introducing the prestressing to coincide with situation *G+P*. If a situation *G+P* is defined in the combination selected above, the concrete stress is checked against the limit value $0.45 f_{c(t)}$ or $0.60 f_{c(t)}$ for this situation depending on the user's specification. The value for $f_{c(t)}$ is also defined in the dialog.

Limiting the Reinforcing and Prestressing Steel Stresses

Reinforcing steel

For reinforcing steel, the limitation of steel stress under the characteristic combination is checked for $0.8\,f_{yk}$ or $1.0\,f_{yk}$ depending on the user's selection according to EN 1992-1-1, Chapter 7.2 (5). The increased limit is permissible for stresses from indirect actions. In this check the reinforcement corresponds to the maximum value from the robustness, crack and bending reinforcement, including a possible increase as a result of the fatigue check. The determination of the strain state is performed at the cracked concrete section. If for beams and design objects tendons with bond are grouted in the check situation, they will be taken into account on the resistance side.

SS EN 1992-1-1:

According to Article 19, the limit $1.0\,f_{yk}$ is generally assumed.

Prestressing steel

For tendons with bond, the limitation of steel stress is checked at the cracked concrete section for beams and design objects and at the uncracked concrete section for area elements. This check is based on the limit $0.75\,f_{pk}$ under the characteristic action combination.

DIN EN 1992-1-1:

The check is carried out for the quasi-continuous combination with the limit $0.65\,f_{pk}$. In addition, the stresses are checked against the minimum of $0.9 \cdot f_{p0,1k}$ and $0.8 \cdot f_{pk}$ under the characteristic combination.

For situations before prestressing and for tendons without bond, the stress $\sigma_{pm0}(x)$ is checked according to Equation (5.43). External tendons are not checked.

Decompression Check

This check is to be carried out for prestressed components of exposure classes XC2-XC4, XD1-XD3 and XS1-XS3 as per Table 7.1N in Chapter 7.3.1 of EN 1992-1-1. According to this, all parts of the tendon with bond or the duct must be located in the overcompressed concrete at a depth of at least 25 mm. The decisive action combination results from the selected exposure class or according to the user specification.

For beams and design objects, the analysis is carried out for stresses resulting from bending and normal force. A cracked section according to Chapter 7.1 (2) is assumed in this analysis in case the tensile stress under the decisive action combination exceeds f_{ctm}. In addition, the rules for stress analysis indicated above apply.

For area sections, an uncracked section is assumed. The 2D concrete stress applied in the direction of the tendon is decisive for the check.

The result is indicated as the 'compression depth' which refers to the shortest distance between the tendon or duct and the tensile zone or section edge. This value is negative if the tendon is in the tensile zone.

DIN EN 1992-1-1:

Table 7.1DE is decisive. The limit state of decompression is maintained if the concrete section around the tendon is under compressive stresses in the range of 100 mm or 1/10 of the section height. The higher range is decisive. The stresses are to be checked in state II.

For structures that are to be designed according to the DAfStb guideline for waterproof components, a compressive stress of 0.5 MN/m² should remain in the component after deducting the loss of prestress according to Chapter 8.4 (2) of the guideline. This can be verified in the graphical representation of the determined check stresses.

OENORM B 1992-1-1:
Table 8AT is decisive.

Minimum Reinforcement for Crack Width Limitation

The minimum reinforcement for crack width limitation is defined in EN 1992-1-1, Chapter 7.3.2. According to 7.3.2 (1), the minimum reinforcement is to be applied in areas where tension is expected. Tension areas can be defined in the section dialog by choosing either an action combination or a restraint (bending, centrical tension). Reinforcing steel layers that are not under tension are also provided with reinforcement in the *symmetrical* and *compression member* design modes. This will not affect the predefined relationships between the individual reinforcement layers.

For profiled sections, each subsection (web or flange) should be checked individually in accordance with Section (2). This cannot be done if any polygonal section geometries are taken into consideration. For this reason, the program always determines the minimum reinforcement based on the entire section. The coefficient k_c is calculated according to user specification either as per Eq. (7.2) or as per Eq. (7.3), optionally different for the top and bottom of the cross-section.

SS EN 1992-1-1:

The permissible crack widths are defined in Article 20, Table D-2, for the quasi-continuous action combination depending on the exposure class, the service life class and the corrosion susceptibility. If the tensile stress does not exceed f_{ctk} / ζ with ζ as per Table D-3, the concrete may be regarded as uncracked. In this case no minimum reinforcement is determined. The program assumes $f_{ctk} = f_{ctk;0.05}(t) = 0.7 \cdot f_{ctm}(t) = 0.7 \cdot f_{ct,eff}$ with $f_{ct,eff}$ according to Equation (7.1).

Determining the minimum reinforcement

Minimum reinforcement $A_{s,min}$ is determined using Equation (7.1) of the standard:

$$A_{s,min} \cdot \sigma_s = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} \tag{7.1}$$

where

A_{ct} is the area of the concrete tensile zone during initial crack formation in state I. To determine the value, the program scales the bending moment of the action combination until the maximum edge stress in state I corresponds to the value $f_{ct,eff}$.

σ_s is the maximum permitted stress in the reinforcing steel reinforcement in relation to the limit diameter of the reinforcing steel.

k is the coefficient for factoring in nonlinearly distributed tensile stresses based on the user's input, which can vary between 0.65 and 1.0 depending on the section height.

DIN EN 1992-1-1:

In case of restraint within the component, these values can be multiplied by 0.8 and for tensile stresses due to restraint generated outside of the component, k = 1.0 shall be assumed.

SS EN 1992-1-1:

According to Article 4a, the value can be assumed to be between 0.50 and 0.90.

$f_{ct,eff}$ is the effective concrete tensile strength at the time of crack formation based on the user's input. The tensile strength is assumed to be f_{ctm} or lower in case the crack formation is expected to occur within the first 28 days. The tensile strength, which depends on the age of the concrete, is defined in Equation (3.4) of Chapter 3.1.2.

DIN EN 1992-1-1:

If it is not certain whether crack formation will occur within the first 28 days, a tensile strength of at least 3 MN/m² for normal concrete and 2.5 MN/m² for lightweight concrete should be assumed.

k_c is the coefficient for consideration of stress distribution prior to crack formation.

$k_c = 1.0$ for tension only

For rectangular sections and webs of box girders or T sections:

$$k_c = 0.4\,(1 - \sigma_c / (k_1 \cdot h \;/\; h^*) / f_{ct,eff}) \le 1 \tag{7.2}$$

For flanges of box girders and T sections:

$$k_c = 0.9 \cdot F_{cr} / A_{ct} / f_{ct,eff} \ge 0.5 \tag{7.3}$$

σ_c is the average concrete stress in the analyzed part of the section with

$$\sigma_c = N_{Ed} / (b \cdot h) \tag{7.4}$$

N_{Ed} is the normal stress in the analyzed part of the section (compressive force positive) under the decisive action combination.

h^* $= \min(h;\ 1.0\ \text{m})$.

k_1 is the coefficient for taking into account the effects of normal force N_{Ed} on the stress distribution:

$k_1 = 1.5$ for compressive normal force

$k_1 = 2\ h^* / (3h)$ for tensile normal force

F_{cr} is the absolute value of the tensile force in the chord directly before crack formation. The tensile force is generated through the integration of tensile stresses within area A_{ct}.

The largest existing bar diameter ϕ_s is specified in the Section dialog (where it is labeled with d_s). It is used in the following equations to determine the limit diameter ϕ_s* as an input value for Table 7.2N:

$\phi_s = \phi_s^* \cdot f_{ct,eff}\ / 2.9 \cdot k_c \cdot h_{cr} / (2(h\text{-}d))$ for bending (7.6N)

$\phi_s = \phi_s^* \cdot f_{ct,eff}\ / 2.9 \cdot h_{cr} / (8(h\text{-}d))$ for centrical tension (7.7N)

where

h is the overall section height.

d is the static effective height up to the centroid of the outermost reinforcement layer.

h_{cr} is the height of the tensile zone directly before crack formation under the decisive action combination.

The limit diameter ϕ_s* and the permissible crack width w_{max} are used to determine the permissible reinforcing steel stress σ_s for Equation (7.1) according to Table 7.2N. The values within the table are interpolated linear, beyond the table they are extrapolated linear for w_k and quadratic for σ_s.

When determining the minimum reinforcement under centrical tension, the steel stress σ_s is calculated with the minimum of the permissible crack width and the maximum of the other quantities, provided they have been defined differently for the cross-section edges.

If the crack width check is to be carried out at the same time, the program will determine whether the specified crack width according to Chapter 7.3.4 is maintained by inserting the calculated minimum reinforcement. If necessary, the minimum reinforcement is increased iteratively until the check limit is reached. The increased reinforcement is indicated by an exclamation mark "!" in the log.

DIN EN 1992-1-1:

The limit diameter ϕ_s* for Table 7.2DE is determined using the following equations:

$\phi_s = \phi_s^* \cdot f_{ct,eff}\ / 2.9 \cdot k_c \cdot k \cdot h_{cr}\ / (4(h\text{-}d)) \geq \phi_s^* \cdot f_{ct,eff}\ / 2.9$ for bending (7.6DE)

$\phi_s = \phi_s^* \cdot f_{ct,eff}\ / 2.9 \cdot k_c \cdot k \cdot h_{cr}\ / (8(h\text{-}d)) \geq \phi_s^* \cdot f_{ct,eff}\ / 2.9$ for centrical tension (7.7DE)

The steel stress σ_s is calculated with the equation from Table 7.2DE and limited to the mean yield strength f_{yk} of the steel layers to be dimensioned.

Based on Chapter 7.3.2 (NA.5), the minimum reinforcement for the crack width limitation in the case of thicker components under centrical restraint can be determined according to Equation (NA.7.5.1). It is not necessary to insert more reinforcing steel as results from Equation (7.1). The rules specified before will be used, if the option is selected by the user, whereas the possibility of lower reinforcement for slowly hardening concrete according to Section (NA.6) will not be used. The evaluation of Figure NA7.1d to determine the effective tensile zone $A_{c,eff}$ is performed with the smallest edge distance d_1 of the reinforcement.

OENORM B 1992-1-1:

Table 7.2N is replaced by Table 8AT. The steel stress σ_s is determined according to Equation (19AT) and limited to the mean yield strength f_{yk} of the steel layers to be dimensioned. The limit diameter is to be modified as follows:

$\phi_s = \phi_s^* \cdot f_{ct,eff}\ / 2.9 \cdot k_c \cdot k \cdot h_{cr}\ / (4(h\text{-}d)) \geq \phi_s^* \cdot f_{ct,eff}\ / 2.9$ (21AT)

For centrical tension $h_{cr} = h / 2$ for each member side is applied.

For members under centrical restraint the minimum reinforcement for the crack width limitation can be determined according to Equation (16AT). This rule will be used, if the option is selected by the user, assuming the smallest edge distance of the reinforcement d_1 for $h\text{-}d$. The program does not take into account the possibility of reducing the reinforcement for slowly hardening concrete.

Special characteristic of prestressed concrete structures

According to the guidelines set forth in Chapter 7.3.2 (3), tendons with bond in the tensile zone may be added to the minimum reinforcement as long as their axis distance to the reinforcing steel layer does not exceed 150 mm. To include the tendons, add the term

$\xi_1 \cdot A_p' \cdot \Delta\sigma_p$

on the left side of Equation (7.1). In this formula

A_p' is the section area of the tendons with bond located in $A_{c,eff}$.

$A_{c,eff}$ is the effective area of the reinforcement according to Figure 7.1. The section after the next describes how $A_{c,eff}$ is determined.

ξ_1 is the adjusted ratio of bond strengths between reinforcing steel and prestressing steel according to Equation (7.5).

$\Delta\sigma_p$ is the stress change in the tendons.

For beams and design objects, the tendon layers with bond can be added using the ξ_1 value specified in the Section dialog as long as they are grouted in the check situation. For area elements, prestressing steel can never be taken into account.

According to Section (4) of Chapter 7.3.2, prestressed concrete components do not require a minimum reinforcement in sections where the absolute value of concrete tensile stress $\sigma_{ct,p}$ under the characteristic action combination and characteristic prestressing is less than $f_{ct,eff}$. This condition is automatically checked by the program.

DIN EN 1992-1-1:
According to Section (4), components with subsequent bond do not require a minimum reinforcement if the absolute value of concrete compressive stress under the characteristic combination on the section edge is greater than 1 N/mm². This condition is also checked by the program.

OENORM B 1992-1-1:
The value is specified as $\sigma_{ct,p} = 0.0$ N/mm².

SS EN 1992-1-1:
The value is specified as $\sigma_{ct,p} = f_{ctk} / \zeta$ with ζ the crack safety factor according to Article 21, Table D-3.
The program assumes $f_{ctk} = f_{ctk;0.05}(t) = 0.7 \cdot f_{ctm}(t) = 0.7 \cdot f_{ct,eff}$ with $f_{ct,eff}$ according to Equation (7.1).

To delimit the areas where no minimum reinforcement is required, the concrete compressive stresses in state I are calculated at the gross cross-section with the mean characteristic prestress. The affected structural areas can be evaluated in the graphical stress representation for the characteristic combination. In the remaining areas, minimum reinforcement is determined if concrete tensile stresses occur in the selected check combination.

Special features for waterproof concrete structures

DIN EN 1992-1-1:
For components that are to be designed according to the waterproof concrete guideline (WU-Richtlinie) of the German Committee for Reinforced Concrete (DAfStb), the permissible crack widths according to Table 2 of the guideline apply. The checks for limiting the crack width shall be lead for the frequent action combination in accordance with Chapter 8.5.1 (1) of the directive.

OENORM B 1992-1-1:
For components that are to be designed according to the oevb guideline for waterproof concrete structures, the permissible crack widths according to Chapter 4.5.1 of the guideline apply depending on the design class according to table 3-2. When using the design model "White tank optimized" ("Weiße Wanne optimiert"), the minimum crack-limiting reinforcement can be avoided if the requirements according to Chapter 4.5.2.2 of the guideline are met and if a minimum reinforcement $A_{s,min}$ is dimensioned as follows per component side and reinforcement direction:

$$A_{s,min} = 0.07 \cdot A_c \cdot f_{ctm} / f_{yk}$$

Crack Width Calculation

The crack width check is performed through direct calculation in accordance with EN 1992-1-1, Chapter 7.3.4, for all sections where tensile stresses in state I occur under the action combination that is based on the exposure class specified in the Table 7.1N. The bar diameter ϕ (d_s in the dialog) of the reinforcing steel reinforcement and the decisive $f_{ct,eff}$ concrete tensile strength are defined in the section dialog.

SS EN 1992-1-1:
The check is performed according to Article 20 for the quasi-continuous action combination.

The program performs the check according to the following steps:

- Determine strain state II under the check combination with the stress-strain curve shown in Figure 3.2. For beams and design objects, all tendons with bond are considered on the resistance side.
- Define the effective area of reinforcement $A_{c,eff}$ shown in Figure 7.1 (see next section), determine the reinforcing steel layers and prestressing steel layers within $A_{c,eff}$.
- Calculate reinforcement level:

$\rho_{p,eff} = (A_s + \xi_1^2 \cdot A_p') / A_{c,eff}$ (7.10)

$\rho_{tot} = (A_s + A_p') / A_{c,eff}$

ξ_1 Bond coefficient according to user specification.

A_s, A_p' Reinforcing steel and prestressing steel within $A_{c,eff}$.

- Determine individually for each reinforcing steel layer:

Difference of the average strain for concrete and reinforcing steel

$\varepsilon_{sm} - \varepsilon_{cm} = [\sigma_s - k_t \cdot f_{ct,eff} / \rho_{p,eff} (1 + \alpha_e \cdot \rho_{p,eff})] / E_s \geq 0.6\ \sigma_s / E_s$ (7.9)

where

$\alpha_e = E_s / E_{cm}$

σ_s is the reinforcing steel stress from strain state II.

DIN EN 1992-1-1:
$\sigma_s = \sigma_{s2} + 0.4 f_{ct,eff} (1/\rho_{p,eff} - 1/\rho_{tot})$ (NA. 7.5.3)
σ_s is limited to f_{yk} in the program
σ_{s2} = Reinforcing steel stress from strain state II

$f_{ct,eff}$ is the effective concrete tensile strength as per specifications.

k_t is the factor for the duration of the load action:
0.6 for short-term and 0.4 for long-term load action.

Maximum crack spacing

$s_{r,max} = k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot \phi / \rho_{p,\,eff}$ (7.11)

where

ϕ is the bar diameter specified by the user.

c is the concrete cover with respect to the longitudinal reinforcement. The concrete cover is set to $d_1 - \phi / 2$ in the program, where d_1 is the smallest axis distance of the reinforcing steel reinforcement of the section edge within $A_{c,eff}$.

k_1 is the coefficient for consideration of the bond properties of the reinforcement. The coefficient is set to 0.8 in the program, which is the recommended value for good bond properties.

k_2 is the coefficient for taking strain distribution into account:
0.5 for bending and 1.0 for pure tension.

k_3, k_4 The recommended national values are $k_3 = 3.4$ and $k_4 = 0.425$.

DIN EN 1992-1-1:
$k_1 \cdot k_2 = 1$, $k_3 = 0$ and $k_4 = 1 / 3.6$
$s_{r,max} \leq \sigma_s \cdot \phi / (3.6 \cdot f_{ct,eff})$.

OENORM B 1992-1-1:
$k_3 = 0$ and $k_4 = 1 / (3.6 \cdot k_1 \cdot k_2) \leq \rho_{p,eff} \cdot \sigma_s / (3.6 \cdot k_1 \cdot k_2 \cdot f_{ct,eff})$ (22AT)
$s_{r,max} = \phi / (3.6 \cdot \rho_{p,eff}) \leq \sigma_s \cdot \phi / (3.6 \cdot f_{ct,eff})$. (23AT)

SS EN 1992-1-1:
$k_3 = 7\ \phi / c$ (Article 22)

If an upper limit for the crack spacing in Equation (7.11) was specified in the section dialog, this allows the special features of Equations (7.13) and (7.14) and sections (4) and (5) of Chapter 7.3.4 to be taken into consideration.

Calculated value of the crack width

$w_k \quad = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm})$ (7.8)

The layer with the largest calculated crack width is shown in the log. If selected in the cross-section dialog, a constant mean steel strain within $A_{c,eff}$ is assumed during calculation.

- For sections completely under tension, the check is performed separately for each of the two effective tensile zones. The maximum value is shown in the log.

If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine a crack reinforcement that is required to maintain the crack width. For that purpose a design is carried out using the decisive check combination for calculating the crack width. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.
The crack width is checked for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check) and saved for graphical representation together with the decisive reinforcing steel stress.

Crack Width Check by Limitation of the Bar Distances

As an alternative to the direct crack width calculation described in EN 1992-1-1, Section 7.3.4, you can choose the simplified check according to Section 7.3.3 (2) through limitation of the bar spacing as shown in Table 7.3N in the cross-section dialog.

The program performs the check according to the following steps:

- Determine strain state II under the check combination defined by the requirement class with the stress-strain curve according to Figure 3.2. For beams and design objects, all tendons in a bond are considered on the resistance side.
- Determine the reinforcing steel stress σ_s for each reinforcement layer. If selected in the cross-section dialog, a constant mean steel stress within $A_{c,eff}$ is assumed for calculating.

  DIN EN 1992-1-1:
  $\sigma_s = \sigma_{s2} + 0{,}4 f_{ct,eff} \, (1/\rho_{p,eff} - 1/\rho_{tot})$ (NA. 7.5.3)
  σ_{s2} = Reinforcing steel stress from strain state II
- Compare the value entered in the dialog ($\max.\ s$) with the table value ($\mathrm{perm.}\ s$), which is derived from the calculated steel stress σ_s and the permissible crack width w_k. In the log, the location with the largest quotient ($\max.\ s$ / $\mathrm{perm.}\ s$) is checked.

If the minimum reinforcement check for limiting the crack width is not selected, the program will automatically determine a crack reinforcement that is required to maintain the permissible bar spacing. For this purpose, a design is carried out with the decisive action combination for the check. The resulting calculated reinforcement is indicated by an exclamation mark "!" in the check log.

The bar spacings are then checked for the final longitudinal reinforcement (maximum from the robustness, crack and bending reinforcement including a possible increase resulting from the fatigue check).

Note

According to Section 7.3.3 (2), the simplified check can only be used in the event of crack formation resulting from mostly direct actions (restraint). In addition, Table 7.3N should only be applied for single-layer tensile reinforcement with d_1 = 4 cm (cf. Zilch, Rogge (2002), p. 277; Fingerloos et al. (2012), p. 109; Book 600 of the DAfStb (2012), p. 127).

OENORM B 1992-1-1:
The method is applicable for single-layer reinforcement with bar spacings according to Table 10AT resp. 11AT. These are valid for concrete covers 25 mm ≤ c_{nom} ≤ 40 mm with bar diameters 8 mm ≤ d_s ≤ 20 mm.

The user is responsible for the evaluation of these requirements.

Determining the Effective Area Ac,eff

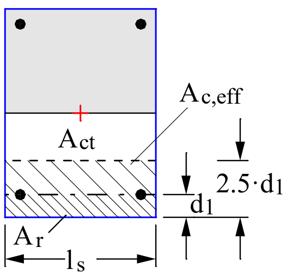
According to EN 1992-1-1, Figure 7.1, the effective area of reinforcement $A_{c,eff}$ defines the area of a rectangular, uniaxially stressed concrete section in which the model assumptions in Book 466 of the German Committee for Reinforced Concrete (DAfStb) are applicable. Although the program can apply this model to any section and stress situation, the user has the responsibility and discretion to do so.

When determining $A_{c,eff}$, the following steps are performed by the program:

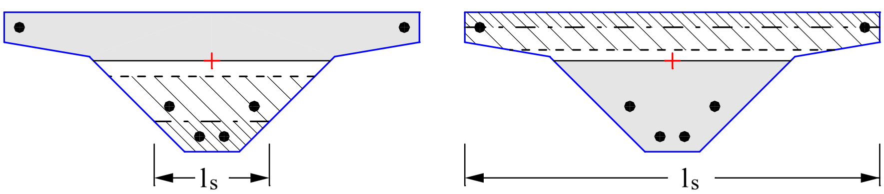
- Determine tensile zone A_{ct} in state I: When calculating the minimum reinforcement, use the stress that led to the initial crack; when calculating the crack width, use the check combination based on the exposure class. In the case of prestressed cross-sections, the specified variation coefficients of the prestressing are taken into account.
- Define the centroid line of the reinforcement as a regression line through the reinforcing steel layers in the tensile zone. In 2D frameworks and for area elements, a horizontal line through the centroid of the reinforcement layers under tension is assumed.
- Determine the truncated residual area A_r to the edge and the sum of section lengths l_s. The average edge distance is then assumed as $d_1 = A_r / l_s$, but not less than the smallest edge distance of the reinforcing steel layers in the tensile zone.
- Shift the centroid line in parallel by $1.5 \cdot d_1$. Assuming $h - d = d_1$, the height of $A_{c,eff}$ is determined as per 7.3.2 (3) by $h_{c,ef} = 2.5 \cdot (h - d) \leq h / 2$. According to DIN EN 1992-1-1 and OENORM B 1992-1-1, Section 7.3.2 (3), this value is limited to $(h - x) / 2$ (x = compressive zone height in state I).

  DIN EN 1992-1-1 and OENORM B 1992-1-1:
  If the minimum reinforcement for thicker components under central restraint is selected in the section dialog, the height of $A_{c,eff}$ is $h_{c,ef} \geq 2.5\ d_1$ according to Figure NA.7.1 d) or Eq. (16AT). In the crack width check, this increase of $h_{c,ef}$ does not apply (see comments in Book 600 for Chapter 7.3.2 (NA.5) and 7.3.4 (2)).
- The resulting polygon is intersected with the tensile zone and then defines the effective area $A_{c,eff}$.
- If all the reinforcing steel layers of the section are under tension, then two zones will be determined; one for the layers above the centroid and the other for layers below the centroid. The area of each zone is limited to $A_c / 2$.

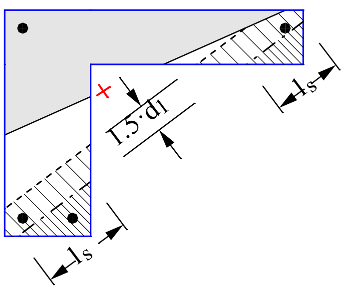
The following illustrations show the effective areas determined by the program for a few representative situations. The last case (edge beam) deviates from the model assumptions in Book 466 to such a degree that it is questionable as to whether it should be used.



Effective area of the reinforcement at a rectangular section under uniaxial bending, normal force with double bending and centrical tension



Effective area of the reinforcement at a bridge section under uniaxial bending



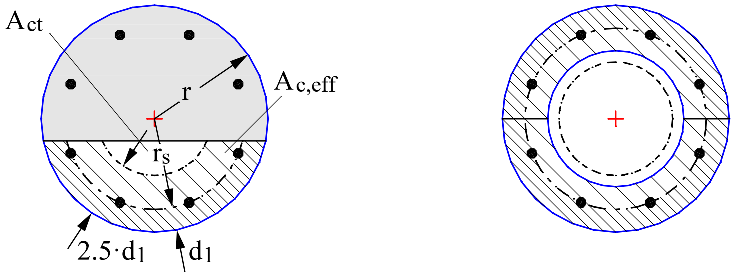
Effective area of the reinforcement at an edge beam under uniaxial bending

Ring-shaped determination of Ac,eff

For circular solid and hollow cross-sections, the cross-section dialog allows that the effective area of the reinforcement $A_{c,eff}$ for checking of the minimum reinforcement and the crack width is determined ring-shaped according to Wiese et al. (2004). This can be used e.g. for considering the specifics of bored piles and spun concrete columns. In order to determine $A_{c,eff}$, the following steps are performed by the program:

- Determine tensile zone A_{ct} in state I: When calculating the minimum reinforcement, use the stress that led to the initial crack; when calculating the crack width, use the check combination based on the exposure class. In the case of prestressed cross-sections, the specified variation coefficients of the prestressing are taken into account.
- Calculate the mean radius r_s of the reinforcing layers within the tensile zone. Assuming the circular radius r of the outer edge, the mean edge distance is determined by $d_1 = r - r_s$.
- The effective area $A_{c,eff}$ is then assumed to be ring-shaped with a width of $2.5 \cdot d_1$ and finally intersected with the tensile zone A_{ct}.
- If all the reinforcing steel layers of the section are under tension, then two ring-shaped zones will be determined; one for the layers above the centroid and the other for layers below the centroid.

The following figures show ring-shaped effective areas exemplarily.



Effective area of the reinforcement at a solid section under bending with normal force as well as a hollow section under centrical tension.

Limiting Deformations

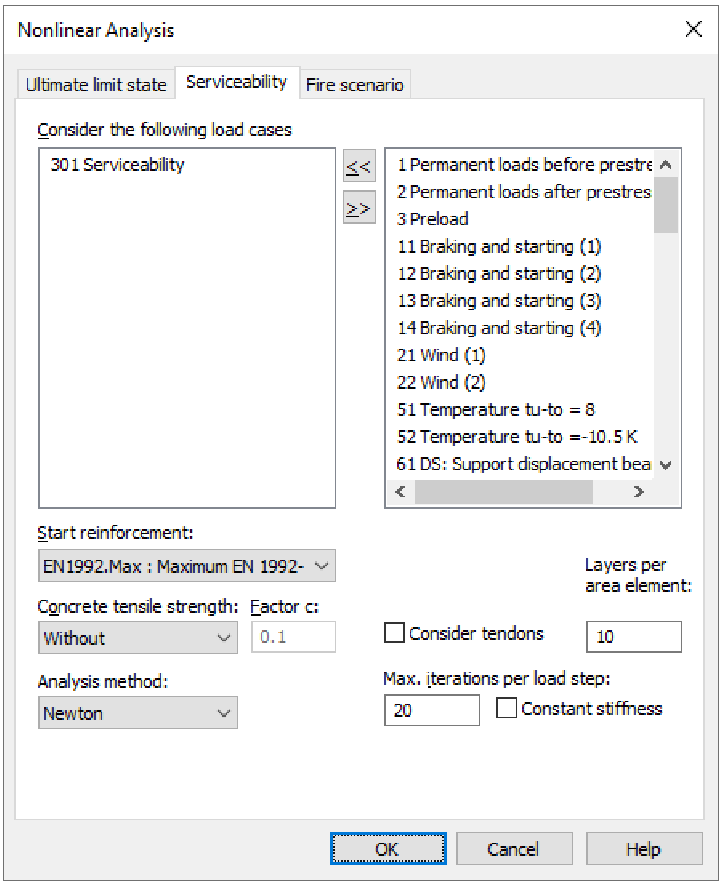
According to EN 1992-1-1, Chapter 7.4.1, the deformations of a component or structure may not impair its proper functioning or appearance. Consequently, a beam, slab or cantilever under the quasi-continuous action combination should not sag more than 1/250th of the span as specified in Section (4). To avoid damage to adjacent components, their deformation should be limited to 1/500th of the span as specified in Section (5).

The standard does not include a method for direct calculation of deformations in accordance with Chapter 7.4.3.

The InfoCAD program system allows you to perform a realistic check as part of a nonlinear system analysis for beam and shell structures that takes geometric and physical nonlinearities into account. The resistance of the tendons within the bond is currently not included in the calculation.

Editing is performed in the following steps:

- Define the check situation using the *Load Group* function in the Load dialog by grouping the decisive individual load cases. The variable loads must first be weighted with the combination coefficients ψ_2 for the quasi-continuous combination.
- Select the check load cases in the *Nonlinear Analysis / Serviceability* dialog in the analysis settings for the FEM or framework analysis.
- Set the reinforcement determined in the ultimate limit state in the *Start reinforcement* selection field (maximum from bending, robustness, crack check and fatigue).
- Perform the FEM or framework analysis to determine the deformations in state II.
- Check the system deformations displayed graphically or in tabular form.



For a detailed description of nonlinear system analysis, refer to the relevant chapter of the manual.

Results

The extremal values for internal forces, support reactions, deformations, soil pressures and stresses are saved for all check situations. The resulting bending, robustness and crack reinforcement, the decisive maximum value and the stirrup and torsion reinforcement are provided for the graphical representation as well.

The log shows the design internal forces and necessary reinforcements, checked stresses or crack widths at each result location. If the permissible limit values are exceeded, they are reported as warnings and indicated at the check location. The detailed log also lists the decisive combination internal forces of all design situations.

Tendon reactions

σ_p, $\Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m²].
d_p, $d_{p,min}$	Depth of the tendons or ducts in the concrete compressive zone in the decompression check [mm].

Stresses for beams and design objects

σ_x	Longitudinal stresses in the decompression and concrete compressive stress checks [MN/m²].
σ_s, $\Delta\sigma_s$	Stresses and stress ranges for reinforcing steel [MN/m²].
σ_p, $\Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m²].
σ_{cd}, $\Delta\sigma_{cd}$	Stresses and stress ranges in the fatigue check for concrete [MN/m²].
$\Delta\sigma_{sb,y}$, $\Delta\sigma_{sb,z}$	Stress ranges for shear reinforcement from Q_y and Q_z [MN/m²].
$\Delta\sigma_{sb,T}$, $\Delta\sigma_{sl,T}$	Stress ranges for shear reinforcement from torsion and longitudinal torsion reinforcement [MN/m²].
σ / σ_{perm}	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

Stresses for area elements

σ_r	Concrete stress in the tendon direction in the decompression check [MN/m²].
σ_{sx}, $\Delta\sigma_{sx}$	Stresses and stress ranges for reinforcing steel in the x direction [MN/m²].
σ_{sy}, $\Delta\sigma_{sy}$	Stresses and stress ranges for reinforcing steel in the y direction [MN/m²].
σ_p, $\Delta\sigma_p$	Stresses and stress ranges for prestressing steel [MN/m²].
$\sigma_{cd,x}$, $\Delta\sigma_{cd,x}$,	Stresses and stress ranges in the concrete fatigue check under longitudinal compression in the x-
$\sigma_{cd,y}$, $\Delta\sigma_{cd,y}$	and y-direction [MN/m²].
$\Delta\sigma_{s,b}$	Stress ranges for shear reinforcement [MN/m²].
σ / σ_{perm}	Stress utilization.
$\Delta\sigma / \Delta\sigma_{perm}$	Stress range utilization.

Bending reinforcement

A_s	Bending reinforcement [cm²] for beams and design objects.
a_{sx}, a_{sy}	Bending reinforcement [cm²/m] for area elements in the x and y direction.
$a_{s\varphi}$	Meridian reinforcement [cm²/m] for axisymmetric shell elements.
$a_{s\upsilon}$	Ring reinforcement [cm²/m] for axisymmetric shell elements.

Reinforcement from lateral force

a_{sbx}, a_{sby}, a_{sb}	Stirrup reinforcement [cm²/m²] of area elements from q_x, q_y and q_r.
$A_{sb.y}$, $A_{sb.z}$	Stirrup reinforcement [cm²/m] of beams and design objects from Q_y and Q_z.
A_{sl} for $a_{sb}=0$	Longitudinal reinforcement [cm²] of area elements.
ΔF_{tdy}, ΔF_{tdz}	Additional tensile force [kN] in the longitudinal reinforcement as a result of lateral force Q_y and Q_z.

Torsional reinforcement

$A_{sb.T}$	Torsional stirrup reinforcement [cm²/m] of beams and design objects from M_x.
$A_{sl.T}$	Torsional longitudinal reinforcement [cm²] of beams and design objects from M_x.

Design values

$V_{Rd,ct}$, $v_{Rd,ct}$	Absorbable design lateral force without shear reinforcement [kN or kN/m].
$v_{Rd,max}$	Absorbable design lateral force of concrete struts for area elements [kN/m].
$V_{Rd,max}$	Absorbable design lateral force of concrete struts for beams and design objects [kN].
$T_{Rd,max}$	Design value of the maximum absorbable torsion moment [kNm].
Q/V_{Rd}+M_x/T_{Rd}	For compact and box sections: $Q\,/V_{Rd,max} + M_x\,/T_{Rd,max}$ DIN EN 1992-1-1: For compact sections: $(Q\,/V_{Rd,max})^2 + (M_x\,/T_{Rd,max})^2$ OENORM B 1992-1-1: For full sections: $(Q\,/V_{Rd,max})^2 + (M_x\,/T_{Rd,max})^2$

Crack width

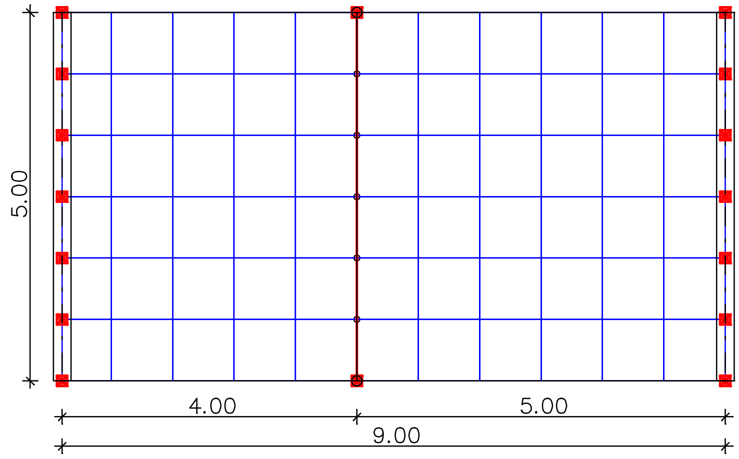
$w_{k.top}$, $w_{k.bottom}$	Computed crack width at the top and bottom of the cross-section separately for the x and y reinforcement direction for area elements.
w_k / w_{per}	Crack width utilization.

Examples

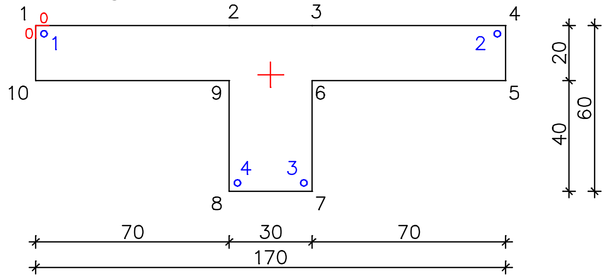
Slab With Downstand Beam

In this example a rectangular slab (d = 20 cm, C30/37-EN, BSt 500 S, exposure class XC2) with a downstand beam will be analyzed. This slab supported with joints will be subjected to its dead load and a traffic load of 10 kN/m².

The checks will be carried out for all possible combinations of load cases. This method is selected in the calculation settings and can take a very long time to complete if there is a large number of load cases.



The following image shows the dimensions of the downstand beam. The axis distance of the reinforcing steel from the section edge is 3 cm. The dead load of the downstand beam is reduced by the portion attributed to the slab.



Design specifications and reinforcing steel description of the slab (section 1):

- Edge distance of the reinforcing steel for the x and y direction of the upper (1st) and lower (2nd) layer: 0.03 m
- Bending design mode: *Standard* with 20% bending reinforcement in secondary direction as per Chapter 9.3.1.1 (2)
- Steel quality: *500S*
- Effective height: 0.17 m
- Strut angle $\cot\Theta$: 1.0.
- Bending tensile reinforcement A_{sl} for the lateral force design: 1.88 cm²

Design specifications of the torsion-flexible downstand beam (section 2):

- Bending design mode: *Standard*
- Steel quality of the stirrups: *500S*
- Shear section:
  Width: 0.30 m
  Effective height: 0.57 m
- Strut angle $\cot\Theta$: 1.0.
- Bending tensile reinforcement A_{sl} for the lateral force design: 2.90 cm²
- Check of the shear joint with default values but joint roughness: *Rough*

EN 1992-1-1 actions

Standard design group

G - Dead load

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

1 Dead load

QN - Imposed load, traffic load

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures
Working load - category A: Residential buildings
Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.3

Load cases 1. Variante, inclusive

2 Traffic span 1
3 Traffic span 2

1. Permanent and temporary situation

Final state

G Dead load
QN Imposed load, traffic load

1. Rare (characteristic) situation

Final state

G Dead load
QN Imposed load, traffic load

1. Quasi-continuous situation

Final state

G Dead load
QN Imposed load, traffic load

Design overview

Se.	Expos. class	Prestress of component	Reinforc. M R B Q T	Fatigue B Q T P C V	Cr. wi.	De-co.	Stress C B P
1	XC4	Not prestressed	x x x x .	. . . . . .	x	.	x x .
2	XC4	Not prestressed	x x x x .	. . . . . .	x	.	x x .

(M) Nominal reinforcement to guarantee robustness.
(R) Nominal reinforcement for crack width limitation.
(B) Flexural reinforcement at ultimate limit state, fatigue and stress check.
(Q) (Nominal-)lateral force reinforcement at ultimate limit state and fatigue.
(T) Torsional reinforcement at ultimate limit and fatigue state.
(P) Prestressing steel at fatigue and stress check.
(C) Concrete comp. stress, concrete at fatigue check under long. compression.
(V) Concrete at fatigue check under lateral force.

Settings for flexural and shear reinforcement

M,N Design mode for bend and longitudinal force:
(ST) Standard, (SY) Symmetrical, (CM) Compression member.
fyk Quality of stirrups.
Theta Angle of concrete truss.
Slabs Beams are designed like slabs.
Asl Given reinforcement according to picture 6.3, increase to maximum.
rhow Factor for minimum reinf. rho.w,min acc. to Chapter 9.3.2(2).
as Factor for bending reinf. of slabs in secondary dir. per 9.3.1.1(2).
Red. Reduction factor of prestress for determining the tensile zone for
distribution of robustness reinforcement for area elements.

Se.	Concr.	Density [kg/m³]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm²] Pic. 6.3 given	Asl [cm²] Pic. 6.3 max	Factor rhow	Factor as	Red. pre-str.
1	C30/37-EN	.	ST	500	1.00	.	1.88	0.00	1.00	0.00	.
2	C30/37-EN	.	ST	500	1.00	.	2.90	.	1.00	.	.

Shear sections

bw.nom Nominal width of the prestressed section according to 6.2.3(6).
h.nom Nominal height of the prestressed section according to 6.2.3(6).
kb, kd Factor to calculate the inner lever arm z from the eff. width bn resp.
from the eff. height d.
z1, z2 Height and width of the core section for torsion.
tef Thickness of the torsion box.
B. Box section; determination of the bearing capacity acc. to Eq. (6.29).

Se.	Width [m] bw	bw.nom	Eff. width bn [m]	kb	Height[m] h	h.nom	Eff.height d [m]	kd	Torsion. section [m] z1	z2	tef	B.
1	1.000	.	.	.	0.200	.	0.170	0.90	.	.	.	.
2	0.300	.	0.270	0.90	0.600	.	0.570	0.90	0.540	0.240	0.060	.

Settings for the shear joint check

Joint roug. Shear joint roughness: very smooth, smooth, rough, indented.
c Factor c acc. to 6.2.5 (2). Values deviationg from the standard are
marked with "!".
dz Distance of the shear joint from the top edge [m].
bi Shear joint width [m].
sigma Stress due to the minimum normal force perpendicular to the joint
(compression negative) [MN/m²].
Dyn. Dynamic or fatigue load acc. to 6.2.5 (5).

Se.	Joint roug.	c	dz [m]	bi [m]	sigma [MN/m²]	Dyn.
2	Rough	0.400	0.200	0.300	0.00	.

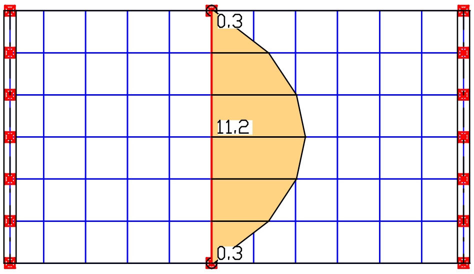
Settings for the check of crack widths

```
ds      Maximal given bar diameter of the reinforcing steel [mm].
max.s   Maximal given bar spacing of the reinforcing steel [mm].
sr,max  Upper limit for the crack spacing from Eq. (7.11) [mm].
Xi1     Bond coefficient of prestressing steel for beam sections.
k       Coefficient for consideration of non-linear distributed tensile stress.
kt      Coefficient for the duration of the load to calculate the crack width.
Fact.   Reduction factor for fctm as per Chapt. 7.3.2 (As) resp. 7.3.4 (wk).
Comb.   Combination for verifying the minimum reinf. (As) and crack width (wk):
        CC, TC, QC = Characteristic, frequent, quasi-continuous combination,
        CT, TT, TB = Centrical tension, tension on top side, tension on bottom,
        CL = Action combination according to exposure class.
Method  Check method for minimum reinf. (kc) and crack width (wk):
kc      Determination of coefficient kc for webs/chords per Eq. (7.2/7.3).
        auto = Eq. (7.2) for rectangular sections, Eq. (7.3) for others.
wk      Calc.   = Direct calculation of crack width as per Chapter 7.3.4,
        Bar sp. = Limiting the bar spacing as per Table 7.3N,
        Cal.(m) = Direct calculation for mean steel strain within Ac,eff,
        Spc.(m) = Lim. the bar spacing for mean steel strain within Ac,eff.
RS      Ring-shaped determination of Ac,eff according to Wiese et al.,
        Beton- und Stahlbetonbau 2004, Issue 4, p 253 ff.

Se.   wmax ds   max  sr   Coefficient   Fact.fctm  Comb.  Method       RS
          [mm]    s  max  Xi1  k    kt    As   wk    As wk  kc    wk
  1   0.30 10     .    .    .   1.00 0.4  1.00 1.00  CL CL  auto  calc.    .
  2   0.30 10     .    .    .   1.00 0.4  1.00 1.00  CL CL  auto  calc.    .
```

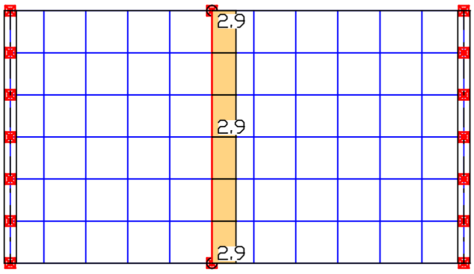
Settings for the check of concrete and steel stresses

```
Sigma.c    Concrete compressive stress in the serviceability limit state.
Sigma.s    Reinforcing steel stress in the serviceability limit state.
(CC),(QC)  Characteristic, quasi-continuous combination.
(TC),(CL)  Frequent combination, combination according to exposure class.

 Se.    fck(t)  per.sigma.c(t)      per.sigma.c     per.sigma.s   Decompression
       [MN/m²]     (CC, QC)          (CC)      (QC)     (CC)      Comb.    Stress
  1       .           .          0.60 fck       .     0.80 fyk      .         .
  2       .           .          0.60 fck       .     0.80 fyk      .         .
```

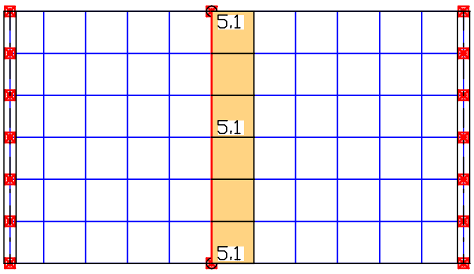
The calculated reinforcements are shown in the illustrations below.



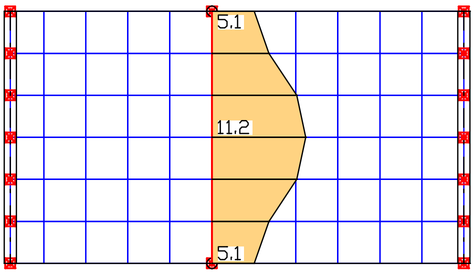
Bending reinforcement A_s of beams from ultimate limit state [cm²]



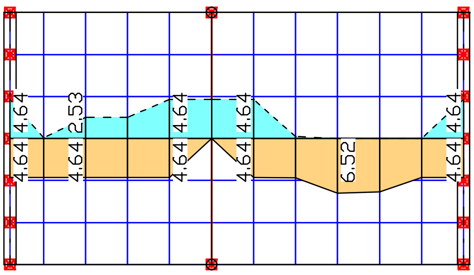
Bending reinforcement A_s of beams for ensuring robustness (ductility) [cm²]



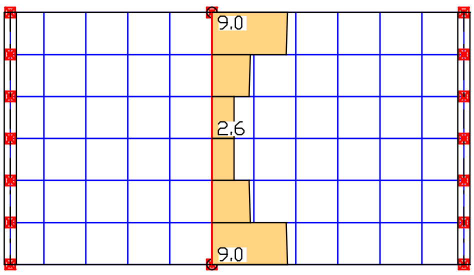
Bending reinforcement A_s of beams from crack width limitation check [cm²]



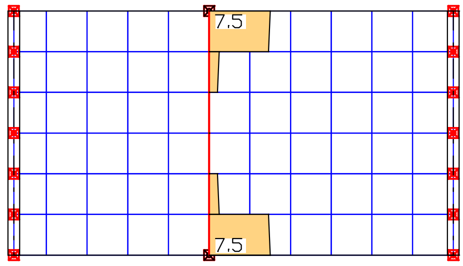
Maximum bending reinforcement A_s of beams [cm²]



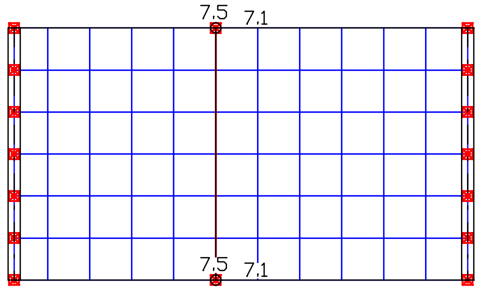
Maximum bending reinforcement a_{sx} of the slab [cm²/m]



Stirrup reinforcement $A_{sb,z}$ of beams from ultimate limit state [cm²/m]



Transverse reinforcement shear joint check A_{sb} of beams from ultimate limit state [cm²/m]



Stirrup reinforcement a_{sb} of the slab from ultimate limit state [cm²/m²] (node mean values)

An excerpt of the detailed log for the midspan of the downstand beam is provided below.

Design of longitudinal reinforcement

```
The calculated requ. reinforcement includes the specified basic reinforcement.

(M) Nominal reinf. for robustness as per EN 1992-2, 6.1 (109) (Charact. C.)
    fctm    Average centric concrete tensile strength [MN/m²]
    zs,t/b  Lever arm of inner strengths top/bottom with zs=0,9*d [m]
    fyk,t/b Strength of longitudinal reinforcement top/bottom [MN/m²]
    max Sc  Maximum concrete edge stress from Charact. C. [MN/m²]
(R) Nominal/requ. reinforcement as per 7.3.2 for crack width limitation
    Increase of reinforcement due to crack width check is marked by "!".
    wmax    Permissible crack width as per specification [mm]
    ds      Maximal given steel diameter [mm]
    k       Coefficient for consideration of non-linear distributed tensile stress
    fct,eff Concrete strength at date of cracking [MN/m²]
    kc      Coefficient to consider stress distribution in tensile zone
            acc. to Eq. (7.2) resp. Eq. (7.3)
    max Sx  Maximal concrete edge stress from action combination [MN/m²]
(B) Design of reinforement at ultimate limit state
    In case of dominant bending, compression reinforcement is marked with "*".
    For section areas acc. to 6.1 (5) the conrecte strain is not limited.
    The minimum reinforcement acc. to 9.2.1.1 and 9.3.1.1 is not determined.
    For compressive members the minimum reinf. acc. to 9.5.2 is considered.
    fck     Concrete strength for design of reinforcement [MN/m²]

Beam 70, x = 0.00 m (Beam length 0.83 m)
Cross-section 2: Polygon - C30/37-EN
Steel 2; Design mode: Standard
(M) fctm=2.9; zs,t/b=0.513/0.513; fyk,t/b=500/500
(R) wmax=0.3; ds=10; kc per Eq. (7.3); k=1; fct,eff=2.9
(B) fck=30

Section properties    A [m²]   ys [m]   zs [m]   Iy [m4]   Iz [m4]   Iyz[m4]
          gross  :     0.460    0.850    0.178    0.0107    0.0828    0.0000
```

1. Characteristic (rare) combination (CC.1): G+QN, Final state

```
Relevant concrete internal forces from 4 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
1    :     0.00     69.95      0.00
2    :     0.00    196.53      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
1    : L1
2    : L1+L2+L3
```

1. Quasi-continuous combination (QC.1): G+QN, Final state

```
Relevant concrete internal forces from 4 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
2    :     0.00    107.92      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : L1+0.30*L2+0.30*L3
```

1. Permanent and temporary comb. (PC.1): G+QN, Final state

```
Relevant concrete internal forces from 8 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
2    :     0.00    284.31      0.00
5    :     0.00     69.95      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : 1.35*L1+1.50*L2+1.50*L3
5    : L1
```

Design of longitudinal reinforcement

```
Reinforcement Nx        My        Mz   max Sc  kc     Ap'   req.As  Situation
Lay. Type    [kN]      [kNm]     [kNm] [MN/m²]        [cm²]   [cm²]
  1  M      0.00     69.95      0.00     .     .       .      0.00  CC.1,1
     R      0.00    107.92      0.00   4.25    .       .      0.00  QC.1,2
     B      0.00     69.95      0.00     .     .       .      0.00  PC.1,5
  2  M      0.00     69.95      0.00     .     .       .      0.00  CC.1,1
     R      0.00    107.92      0.00   4.25    .       .      0.00  QC.1,2
     B      0.00     69.95      0.00     .     .       .      0.00  PC.1,5
  3  M      0.00    196.53      0.00   7.73    .       .      1.44  CC.1,2
     R      0.00    107.92      0.00   4.25  0.50      .      2.53  QC.1,2
     B      0.00    284.31      0.00     .     .       .      5.60  PC.1,2
  4  M      0.00    196.53      0.00   7.73    .       .      1.44  CC.1,2
     R      0.00    107.92      0.00   4.25  0.50      .      2.53  QC.1,2
     B      0.00    284.31      0.00     .     .       .      5.60  PC.1,2
```

Design of shear reinforcement

```
The percentage of nominal reinforcement acc. to Eq. (9.5N) is considered.

Ac         Section area for calculating the concrete stress from long. force [m²]
bw         Effective width for calculation of shear stresses from Qz and Mx [m]
bn         Statically effective width for shear design using Qy [m]
kb         Factor to calculate the inner lever arm from bn
h          Effective height for calculation of shear stresses from Qy and Mx [m]
d          Statically effective height for shear design using Qz [m]
kd         Factor to calculate the inner lever arm from d
Angle      Angle cot Theta between the compressive strut and the beam axis
Asl giv.   Chargeable longitudinal reinf. acc. to Fig. 6.3 [cm²]
rhow,min   Minimal percentage of lateral reinforcement acc. to Eq. (9.5N)
Qy, Qz     Lateral forces for design in y- and z-direction [kN]
VRdc       Absorbable lat. force without lat. reinf. per 6.2.2 (1) [kN]
VRdmax     Absorbable lateral force of comp. struts per 6.2.3 (3) [kN]
z          Inner lever arm z=kb*bn resp. z=kd*d [m]
Asb.y,z    Req. stirrup reinforcement from Qy resp. Qz [cm²/m]
Asl        Req. longitudinal reinf. acc. to Fig. 6.3 [cm²] for req.Asb
Delta Ftd  Tensile force in long. reinf. from lateral force as per Eq. (6.18)

Beam 70, x = 0.00 m (Beam length 0.83 m)
Cross-section 2: Polygon - C30/37-EN
bw/bn/kb=0.3/0.27/0.9; h/d/kd=0.6/0.57/0.9
Ac=0.46; fyk=500; Asl giv./max=2.9/0; rhow,min=1*(0.08*fck½/fyk)
```

1. Permanent and temporary comb. (PC.1): G+QN, Final state

```
Relevant concrete internal forces from 8 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]   Mx[kNm]    Qy[kN]    Qz[kN]
2    :     0.00    284.31      0.00      0.00      0.00    -30.79

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : 1.35*L1+1.50*L2+1.50*L3
```

Check of the shear reinforcement and the compressive struts

```
Action        z          Q/     Asb.y   Asb.z   Asb.T   Asl.T   Asl
max          [m]    Angle VRdc  [cm²/m] [cm²/m] [cm²/m]  [cm²]  [cm²]  Situation
Qy       :  0.24    1.00  0.00      .       .       .       .      .   -,-
Qz       :  0.51    1.00  0.47      .     2.63      .       .     2.90  PC.1,2

Action        z          Qy/     Qz/     Mx/     Q/VRd+    Delta Ftd
max          [m]    Angle VRdmax  VRdmax  TRdmax  Mx/TRd          [kN]  Situation
Qy       :  0.24    1.00  0.00      .       .       .             0.00  -,-
Qz       :  0.51    1.00   .       0.04     .       .            15.40  PC.1,2
```

Shear joint check

```
bi         Width of the joint [m]
sigma.n    Stress perpendicular to the joint acc. to user specifications.
           Negative for compression [MN/m²]
Roughness  Joint roughness (very smooth, smooth, rough, indented)
c, mue     Factors dependent on joint roughness according to 6.2.5(2)
z          Inner lever arm according to shear section [m]
beta       Ratio of longitudinal force in the new concrete and total longitudinal
           force in compression or tension zone. If the strain state cannot be
           clearly determined, 1.0 is assumed on the save side (marking *)
vEdi       Design value of shear stress in the joint acc. to Eq. (6.24) [MN/m²]
vRdi       Design value of shear resistance in the joint acc. to Eq. (6.25)
           [MN/m²]
vEdi/vRdi  Utilization rate
req. Asb   Area of the required transverse reinforcement [cm²/m]

Beam 70, x = 0.00 m (Beam length 0.83 m)
Cross-section 2: Polygon - C30/37-EN
bi = 0.3 m; sigma.n = 0 MN/m²
Roughness = Rough; c = 0.4; mue = 0.7
```

1. Permanent and temporary comb. (PC.1): G+QN, Final state

```
Relevant concrete internal forces from 8 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]   Qz[kN]
2    :     0.00    284.31      0.00   -30.79

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : 1.35*L1+1.50*L2+1.50*L3
```


Shear joint check

```
  z   beta    vEdi     vRdi    vEdi/  req. Asb
 [m]  [-]    [MN/m²]  [MN/m²]  vRdi    [cm²/m]   Situation
 0.51 1.00    0.2001   0.5413   0.37      0.00   PC.1,2
```

Check of crack widths

```
The check is led by direct calculation of the crack width.
The final long. reinforcement as the maximum from robustness, crack and bending
reinf. incl. a possible increase resulting from the fatigue check is decisive.

(CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination

wmax     Permissible crack width as per specification [mm]
ds       Maximal given steel diameter [mm]
fct,eff  Concrete strength at date of cracking [MN/m²]
Sigma.c  Maximal concrete edge stress in state I [MN/m²]
wk       Calculated value of crack width as per 7.3.4 [mm]
sr,max   Calculated / given maximal crack spacing as per 7.3.4 (3) [mm]
Ac,eff   Effective region of reinf. [m²] acc. to Fig. 7.1
As,eff   Reinforcing steel within Ac,eff [cm²]
Ap,eff   Prestressing steel with bond within Ac,eff [cm²]
Sigma.s  Reinf. steel stress in state II [MN/m²]
kt       Coefficient for the duration of load as per 7.3.4 (2)

Beam 70, x = 0.00 m (Beam length 0.83 m)
Cross-section 2: Polygon - C30/37-EN
wmax=0.3; ds=10; fct,eff=2.9; kt=0.4

Section properties    A [m²]   ys [m]   zs [m]   Iy [m4]   Iz [m4]   Iyz[m4]
          gross  :     0.460    0.850    0.178    0.0107    0.0828    0.0000
```

1. Quasi-continuous combination (QC.1): G+QN, Final state

```
Relevant concrete internal forces from 4 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
2    :     0.00    107.92      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : L1+0.30*L2+0.30*L3
```

Check of crack width for reinf. layer 3 (bottom)

```
Nx        :     0.00 kN      As,eff  :  11.19  cm²
My        :   107.92 kNm     Ap,eff  :    .    cm²
Mz        :     0.00 kNm     Ac,eff  :   0.023 m²
Sigma.c   :     4.25 MN/m²   Sigma.s : 175.61  MN/m²
Situation : QC.1,2           sr,max  : 119.17  mm
                             wk      :   0.09  wmax 0.30 mm
```

Check of concrete compressive stress

```
For the check, a cracked concrete section (II) is assumed if the tensile stress
from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section
(I) is used. If the strain is not absorbable on cracked section, (I*) is marked.

fck           Characteristic compressive concrete strength [MN/m²]
Sigma.x,min   Total maximal longitudinal compressive stress [MN/m²]
Sigma.x,per   = 0,60*fck for Charact. C. (CC) as per 7.2 (2)
top, bottom   Position of the edge point: above, below of centre

Beam 70, x = 0.00 m (Beam length 0.83 m)
Cross-section 2: Polygon - C30/37-EN
0.6*fck=18

Section properties    A [m²]   ys [m]   zs [m]   Iy [m4]   Iz [m4]   Iyz[m4]
          gross  :     0.460    0.850    0.178    0.0107    0.0828    0.0000
```

1. Characteristic (rare) combination (CC.1): G+QN, Final state

```
Relevant concrete internal forces from 4 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
1    :     0.00     69.95      0.00
2    :     0.00    196.53      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
1    : L1
2    : L1+L2+L3
```

Check of compressive stress in concrete for the Characteristic (rare) combination

```
Side    Se.-  min Sigma.x  per. Sigma.x  Period   Situation
        Pnt.      [MN/m²]       [MN/m²]
top       1  (II)  -6.58        -18.00  Final    CC.1,2
bottom    7  (II)   0.00        -18.00  Final    CC.1,1
```

Check of steel stress

```
For the check, a cracked concrete section is assumed.

Type S  Long. reinf. from N and M, layer number, Charact. C. (CC)
fck     Concrete strength to determine the strain state [MN/m²]
Sigma.s,per = 0.80 * fyk resp. 1.0 * fyk (CK) as per 7.2 (5)

Beam 70, x = 0.00 m (Beam length 0.83 m)
Cross-section 2: Polygon - C30/37-EN
fck=30; Steel 2; 0.8*fyk,t/b=400/400

Section properties    A [m²]   ys [m]   zs [m]   Iy [m4]   Iz [m4]   Iyz[m4]
          gross  :     0.460    0.850    0.178    0.0107    0.0828    0.0000
```

1. Characteristic (rare) combination (CC.1): G+QN, Final state

```
Relevant concrete internal forces from 4 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
1    :     0.00     69.95      0.00
2    :     0.00    196.53      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
1    : L1
2    : L1+L2+L3
```

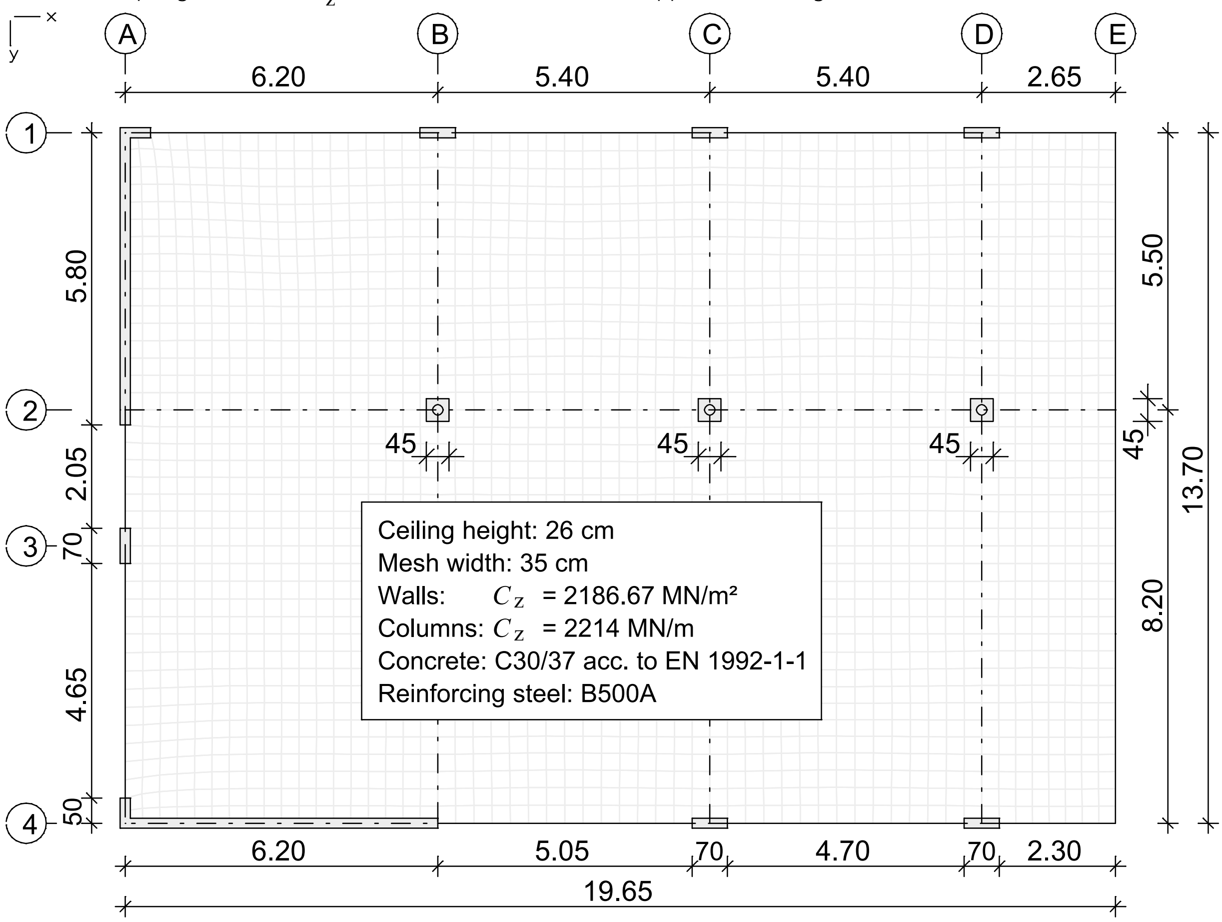
Check of steel stress

Steel Type	No.	Nx [kN]	My [kNm]	Mz [kNm]	As [cm²]	Sigma.s [MN/m²]	per. [MN/m²]	Situation
S	1	0.00	69.95	0.00	0.00	.	400.00	CC.1,1
S	2	0.00	69.95	0.00	0.00	.	400.00	CC.1,1
S	3	0.00	196.53	0.00	5.60	319.92	400.00	CC.1,2
S	4	0.00	196.53	0.00	5.60	319.92	400.00	CC.1,2

Flat Ceiling With Cantilever

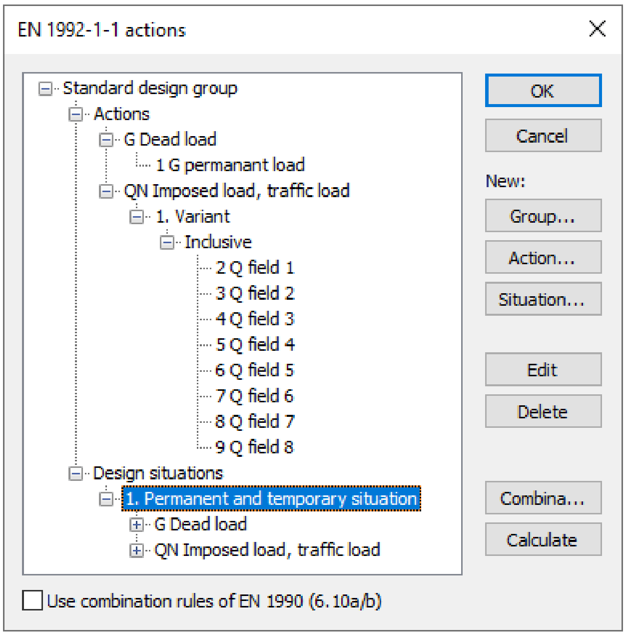
This example is a ceiling slab corresponding to Example 18 in the book '*Beispiele zur Bemessung nach Eurocode 2 – Band 2: Ingenieurbau*'. The hinged slab is loaded with permanent loads and a traffic load applied span by span. The load cases are calculated linearly-elastically, combined to minimum and maximum design internal forces according to the permanent and temporary design situations and designed for the ultimate limit state. Furthermore, elastic calculations are performed for a deformation check.

The vertical spring stiffnesses C_z of the walls and columns are applied according to the literature.

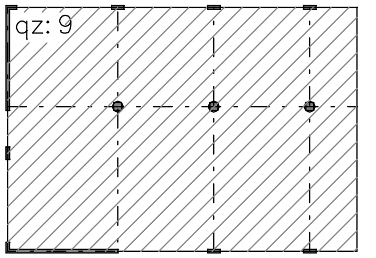


Element system with dimensions [m]

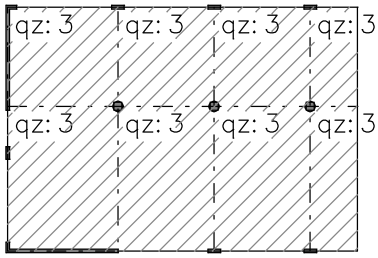
Design according to EN 1992-1-1



Loads



Load case 1, G permanent load [kN/m²]



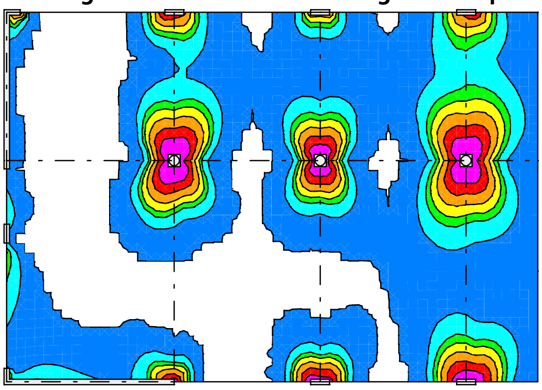
Load case 2 to 9, Q field 1 to 8 [kN/m²]

Reinforcement for area elements

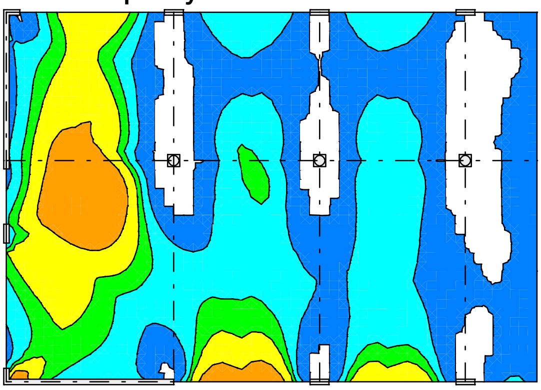
No.	Lay.	Qual.	d1x [m]	d2x [m]	asx [cm²/m]	d1y [m]	d2y [m]	asy [cm²/m]	as fix	Roll-ing
1	1	500M	0.045		0.000	0.035		0.000		Warm
	2	500M		0.035	0.000		0.025	0.000		Warm

as Base reinforcement
d1 Distance from the upper edge
d2 Distance from the lower edge
The z axis of the element system points to the lower edge

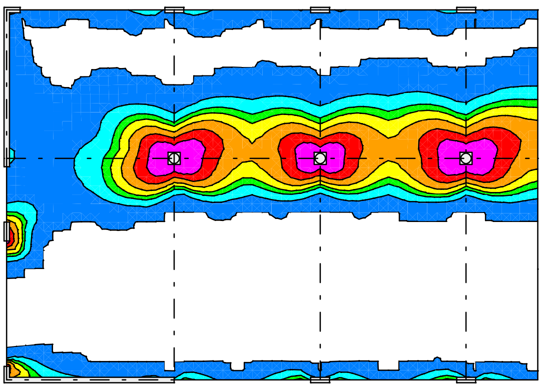
Bending reinforcement from design of the permanent and temporary situation



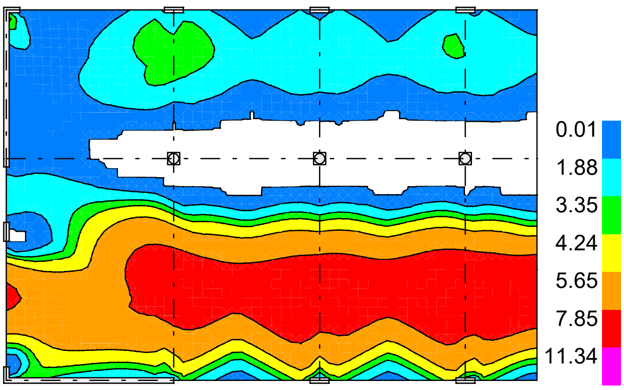
Bending reinforcement upper layer $a_{sx.1}$ [cm²/m]



Bending reinforcement lower layer $a_{sx.2}$ [cm²/m]



Bending reinforcement upper layer $a_{sy.1}$ [cm²/m]



Bending reinforcement lower layer $a_{sy.2}$ [cm²/m]

Deformation u_z

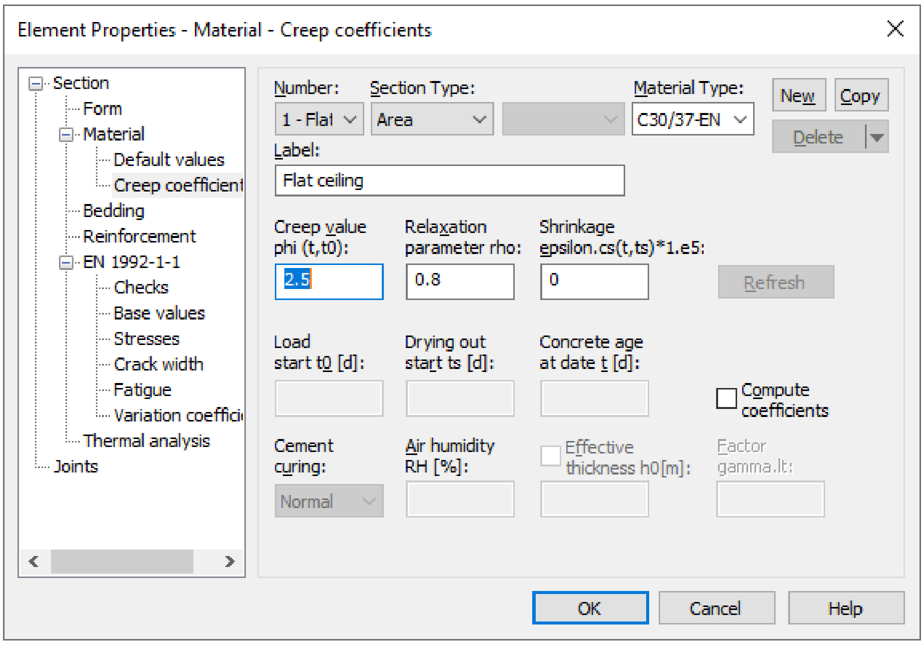
The combination coefficient ψ_2 of the quasi-continuous situation is assumed to be 0.3 in this example. For the calculation of the creep deformation a creep-generating permanent load case has to be defined (here superposition load case number 11).

Load data load case 11: G+0.3·Q with elastic material behavior

Superposition of results (SUP)

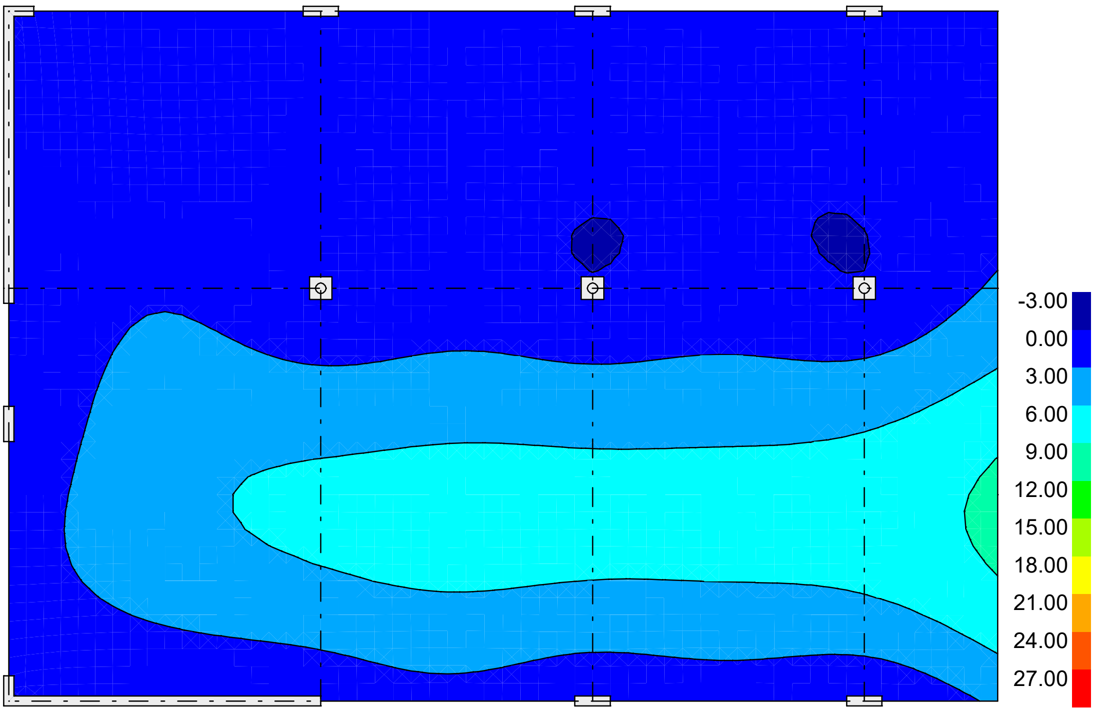
No.	load case from	to	weighting
1	1	1	1.000
2	2	9	0.300

The concrete creep is then calculated in load case 12 (load type *Creep and shrinkage*). In addition to the data in the load case, the creep and shrinkage coefficients of the concrete material specified in the cross-section are required. Since the shrinkage coefficient has no meaning for slab elements, it was set to zero in this example.Alternatively, the coefficients could have been calculated via the dialog according to the formulas of the standard.

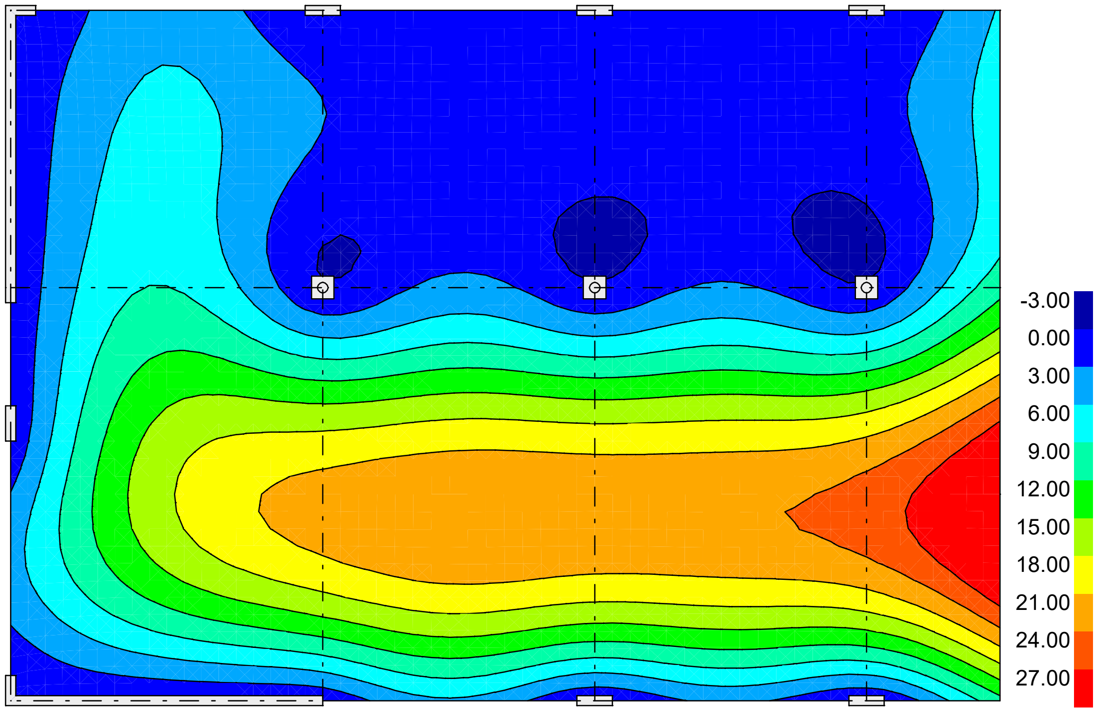


The internal forces due to creep were not taken into account in the design, since they generally remain insignificant for all elements with identical creep coefficients. However, relevant deformations due to creep are to be expected.

Below are some of the results.



Load case 11 (G+0.3·Q): Color gradient of the deformations u_z [mm] at the start of loading

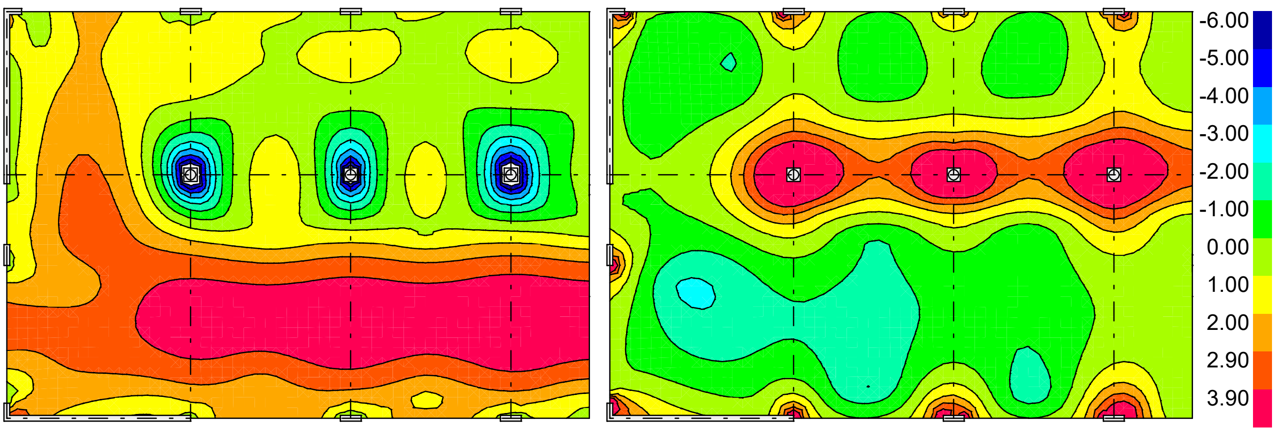


Load case 13: Color gradient of the deformations u_z [mm] at time t_∞

In summary, the following maximum deformations result in state I:

Load case		max u_z [mm]	
		Calculation	**Literature**
11	G+0.3·Q with elastic material behavior	10	10
12	Creeping (φ=2,5 by LC 11) with elastic material behavior	23	
13	Total (load case 11 + load case 12)	33	

Principal tensile stresses σ_1



Load case 13: Color gradient of the principal tensile stresses $\sigma_{1,bottom}$ and $\sigma_{1,top}$ [MN/m²]

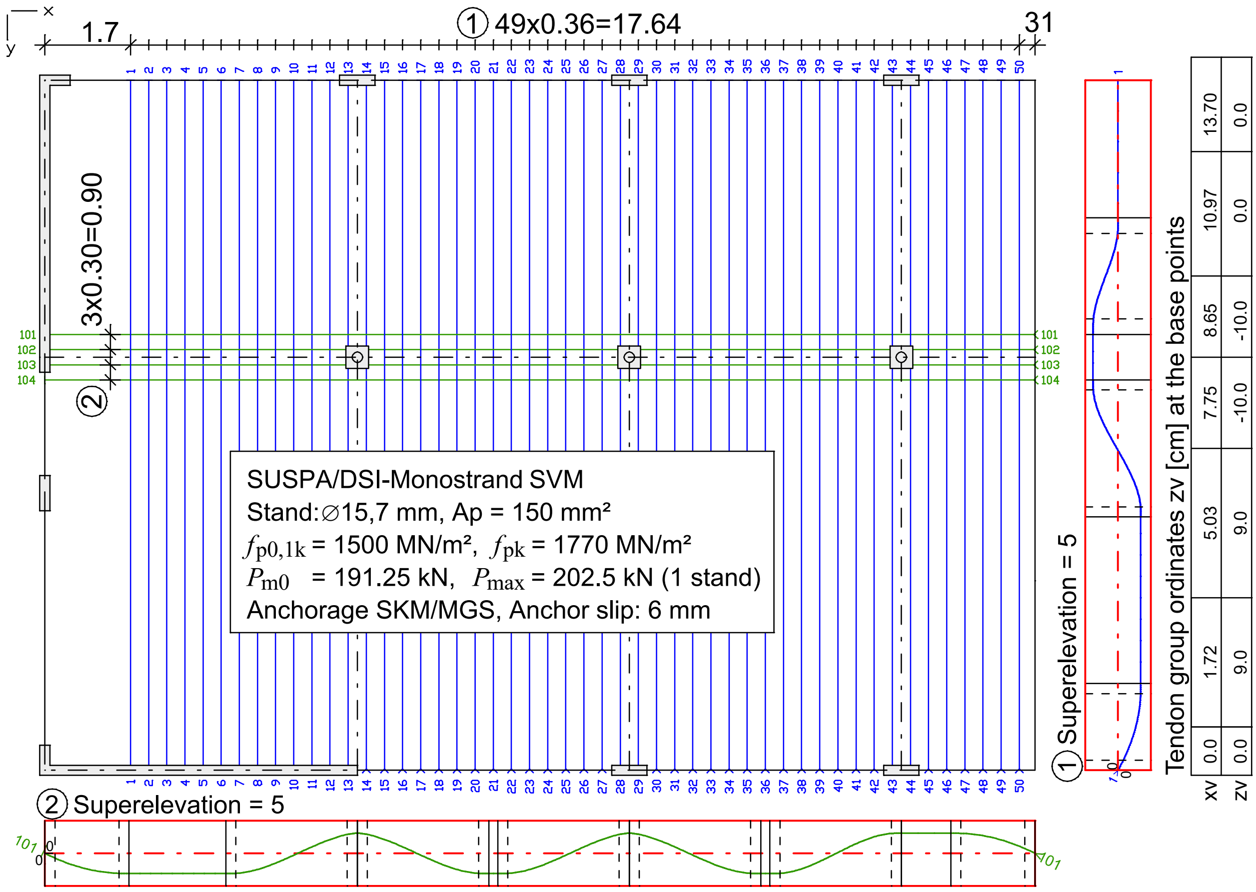
If Since the principal tensile stresses σ_1 exceed the concrete tensile strength f_{ctm} of 2.9 MN/m in a large area, the calculation in state I is not suitable for the deformation check. In the manual section for the nonlinear system analysis, the deformations are therefore examined in detail in state II for this example.

Flat Ceiling With Cantilever and Prestressing

This example is a variant of the previous example with prestressing without bond. For comparison, the required concrete reinforcement according to EN 1992-1-1 and the linear-elastic deformations are also determined. The prestressing is primarily intended to reduce the deflection of the slab.

Since a prestressed slab also results in normal forces in the area, shell elements must be used and care must be taken to ensure that the horizontal supports are as free of restraints as possible. Otherwise, large tensile stresses would result from the shrinkage of the concrete.

Monostrands of 7-strand prestressing steel St 1570/1770 with a corrosion protection system in free tendon position are used. The 50 tendons (1) in y-direction are uniformly distributed single monostrands. The four tendons (2) in the x-direction are double bundled monostrands.



Tendon group ordinates zv [cm] at the base points

xv	0.00	1.67	3.59	6.20	8.81	8.99	11.60	14.21	14.39	17.00	17.98	19.65
zv	0.0	8.0	8.0	-8.0	8.0	8.0	-8.0	8.0	8.0	-8.0	-8.0	0.0

System with arrangement of monostrands in free tendon position with their high and low points

Loads

In addition to the load cases of the previous example, a prestressing load case (load type *Prestressing*) is required.The creep-generating permanent load case 11 is supplemented by the prestressing load case. Since the tendons have no bond with the concrete, the shrinkage of the concrete of $\varepsilon_{s,\infty}$ = -40·10^{-5} assumed in the literature leads to an additional anchor slip of 7.9 mm in x direction and 5.5 mm in y direction. Thus, at time t_∞, neglecting friction, a prestressing force of 178 kN per tendon (1) and 359 kN per tendon (2) results. The prestressing at an earlier time is not investigated in this example.

Load data load case 10: P prestressing

No.	
1	Prestressing (VSPG)

Load data load case 11: G+P+0.3·Q with elastic material behavior

Superposition of results (SUP)

No.	load case from	load case to	weighting
1	1	1	1.000
2	10	10	1.000
3	2	9	0.300

Design according to EN 1992-1-1

The design-relevant actions are supplemented by the prestressing and the creep load case.

EN 1992-1-1 actions

Standard design group

QN - Imposed load, traffic load

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures
Working load - category A: Residential buildings
Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.3

Load cases 1. Variant, inclusive	
2	Q field 1
3	Q field 2
4	Q field 3
5	Q field 4
6	Q field 5
7	Q field 6
8	Q field 7
9	Q field 8

G - Dead load

Gamma.sup / gamma.inf = 1.35 / 1

Load cases	
1	G permanant load

P - Prestressing

Gamma.sup / gamma.inf = 1 / 1

Load cases internal prestressing	
10	P prestressing

CSR1 - Creep, shrinkage, relaxation

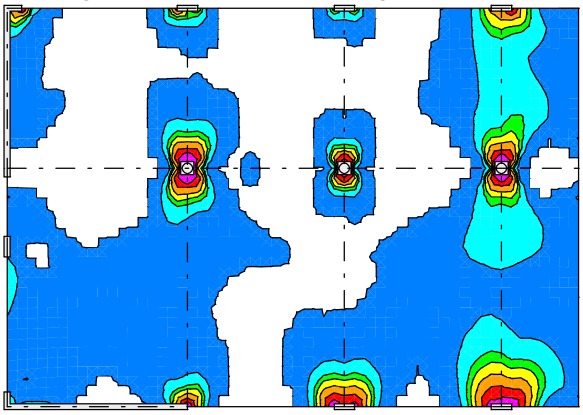
Load cases	
12	Creeping (φ=2.5 by LC 11) with elastic material beh...

1. Permanent and temporary situation

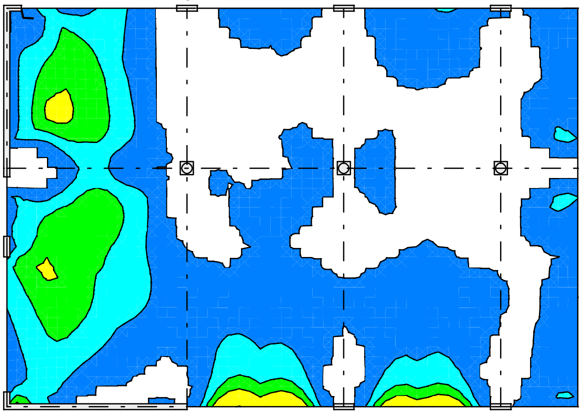
Final state

G	Dead load
P	Prestressing
CSR1	Creep, shrinkage, relaxation
QN	Imposed load, traffic load

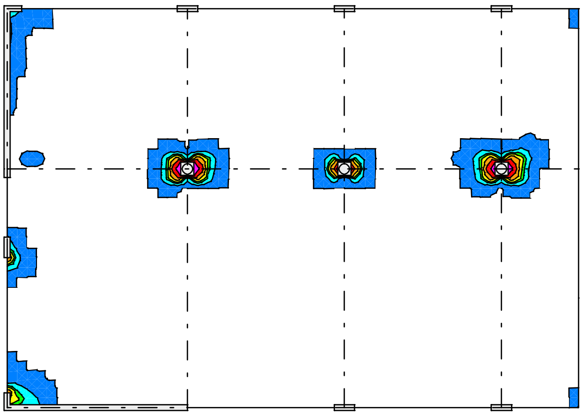
Bending reinforcement from design of the permanent and temporary situation



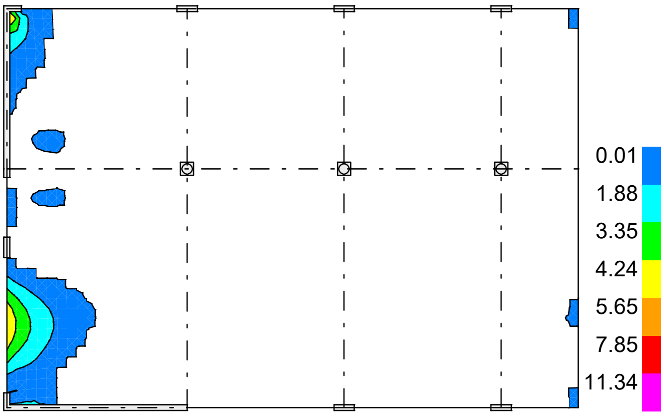
Bending reinforcement upper layer $a_{sx.1}$ [cm²/m]



Bending reinforcement lower layer $a_{sx.2}$ [cm²/m]

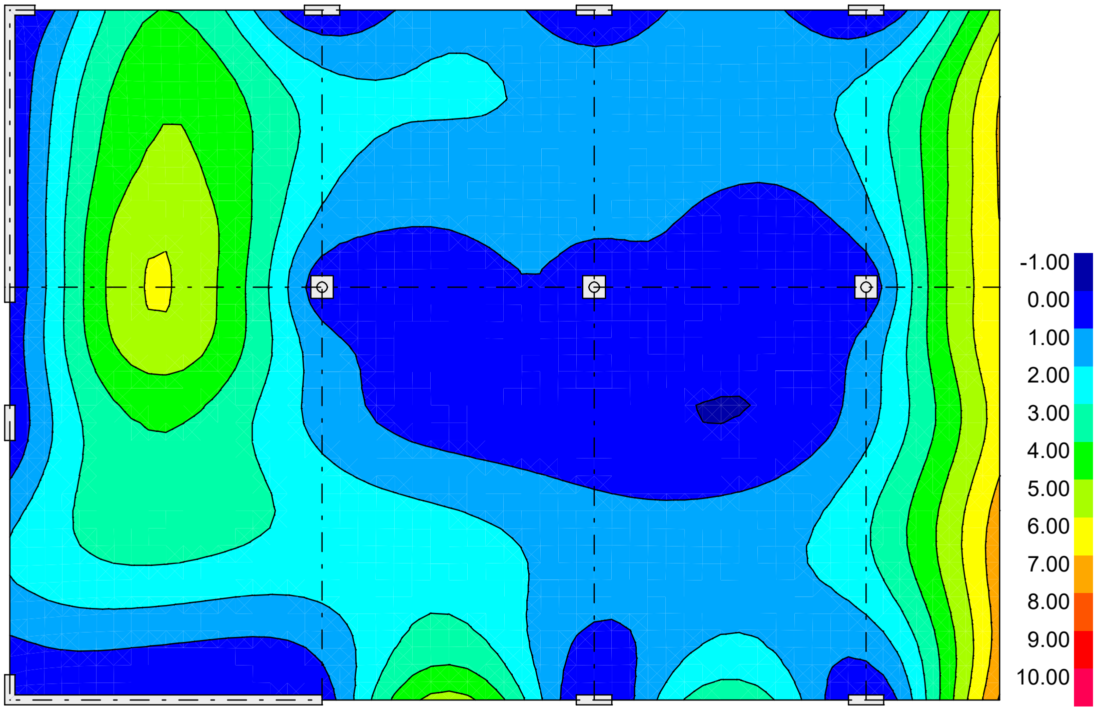


Bending reinforcement upper layer $a_{sy.1}$ [cm²/m]



Bending reinforcement lower layer $a_{sy.2}$ [cm²/m]

Deformation u_z

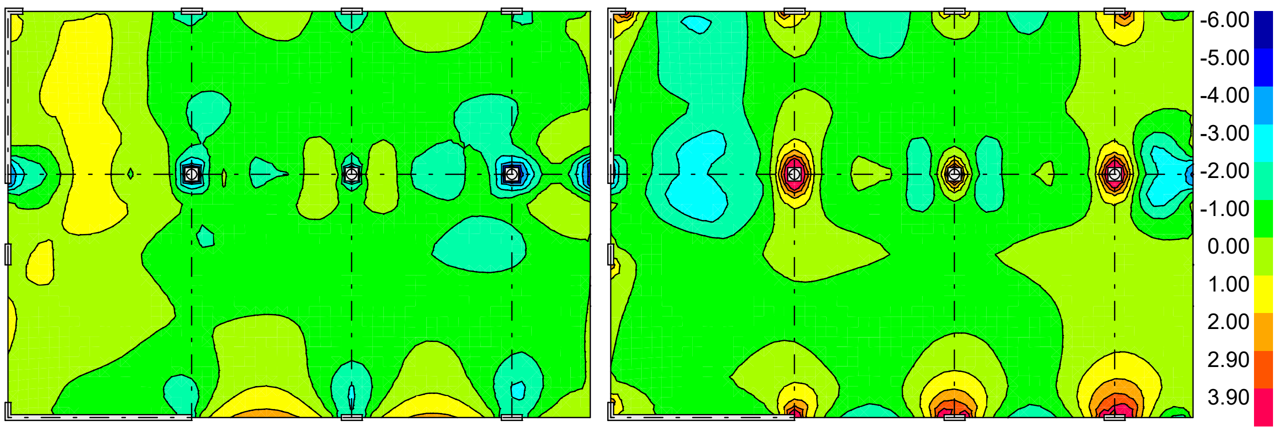


Load case 13: Color gradient of the deformations u_z [mm] at time t_∞

In summary, the following maximum deformations result in state I compared to the previous example without prestressing:

Load case		**max u_z [mm]** with prestress	**max u_z [mm]** without prestress
11	G+P+0.3·Q with elastic material behavior	3	10
12	Creeping (φ=2,5 by LC 11) with elastic material behavior	5	23
13	Total (load case 10 + load case 11)	8	33

Principal tensile stresses σ_1



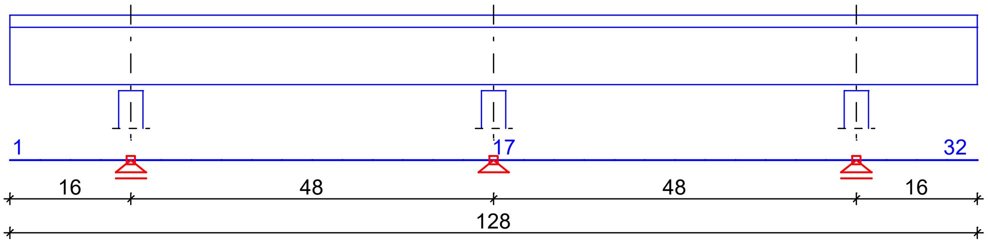
Load case 13: Color gradient of the principal tensile stresses $\sigma_{1,\text{bottom}}$ and $\sigma_{1,\text{top}}$ [MN/m²]

Since the principal tensile stresses σ_1 exceed the concrete tensile strength f_{ctm} of 2.9 MN/m² only in small areas, an investigation in state II is not performed.

Prestressed Roof Construction

This example involves the wide-spanned roof construction of an entrance hall that is represented as a continuous girder over two spans with a double-sided cantilever. A T-beam is selected as the section. The figure below shows the system in longitudinal and lateral section view.

Limited prestressing with subsequent bond is applied to the roof construction in the longitudinal direction. Prestressing in the lateral direction is not applied for reasons of economy. The construction is designed to meet exposure class XC1. According to Table 7.1N of the EN 1992-1-1, a decompression check is not necessary for this class.

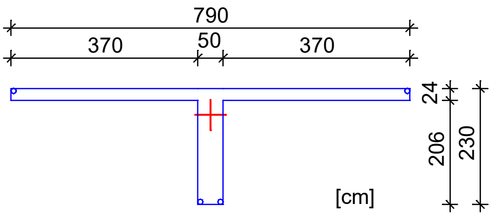


Static system and dimensions [m]

Material

Concrete	*C45/55-EN*
Reinforcing steel	*BSt 500*, axis distance from edge 5 cm

Section



Prestressing steel and prestressing system

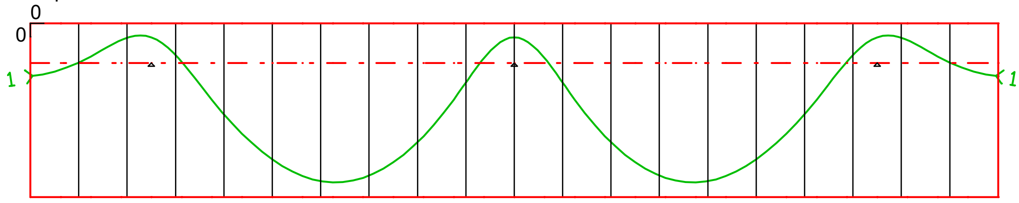
Prestressing steel quality	St 1500/1770
Certification of the prestressing system	EC2
Number of tendons in the bundle	4
Section surface A_p	1800 mm²
E-modulus of the prestressing steel	195000 MN/m²
0.1% strain limit (yield strength) of the prestressing steel $f_{p0.1k}$	1500 MN/m²
Tensile strength of the prestressing steel f_{pk}	1770 MN/m²
Permissible prestressing force of a tendon P_{m0}	2295 kN
Prestressing loss from relaxation of prestressed steel	4.5 %
Friction coefficients when prestressing and releasing μ	0.2
Unintentional deviation angle of a tendon β'	0.3 °/m
Slippage at prestressing anchor	6 mm
Duct diameter d_h	82 mm

Variation coefficients of the internal prestressing

Construction stage (r_{sup} / r_{inf})	1.1 / 0.9
Final state (r_{sup} / r_{inf})	1.1 / 0.9

The tendon guide is shown in the next figure. 4 bundled tendons are arranged such that they stretch across the entire girder length and are prestressed at both girder ends. The prestressing system, prestressing procedure and prestressing curve for a tendon group are also shown.

Tendon groups in beam series view 1, [-16.00/0.00/0.00] - [112.00/0.00/0.00] / [-16.00/0.00/1.00]
Superelevation = 10



Tendon group ordinates zv [cm] at the base points

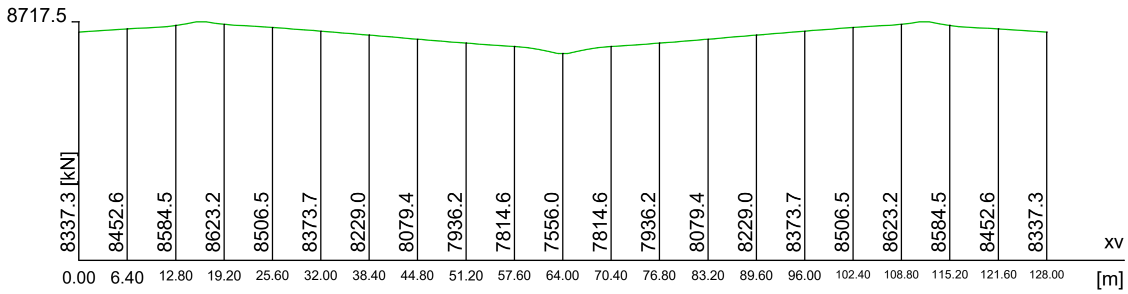
xv	0.00	6.40	12.80	19.20	25.60	32.00	38.40	44.80	51.20	57.60	64.00	70.40	76.80	83.20	89.60	96.00	102.40	108.80	115.20	121.60	128.00
1	70.0	52.0	19.2	42.0	120.2	180.1	208.8	201.5	157.4	78.3	18.5	78.3	157.4	201.5	208.8	180.1	120.2	42.0	19.2	52.0	70.0

Force function of tendon group 1 (4 tendon(s), l = 128.41 m)

Prestressing system 1 - Example. Certification according to EC2.
Pm0 = 2295.0 kN, Ap = 1800.0 mm², μa = 0.20, Angle ß' = 0.30 °/m
E-Modulus= 195000 MN/m², Ah = 5281.0 mm², μn = 0.20, Slippage = 6.00 mm

Prestressing procedure 1 - Example

Pre. anchor :	Start	End
Normal. force :	1.000	1.000
Pre. force [kN]:	2295.0	2295.0
Extension [mm]:	694.5	72.8



Tendon guide and prestressing curve in the longitudinal section (4 tendons).

Loads

Load case 1	Dead load
Load case 2	Additional dead load: q=11.06 kN/m
Load case 3	Snow load: q=7.90 kN/m
Load case 10	Prestressing
Load case 15	Creep-generating permanent load: Dead load, additional dead load and prestressing
Load case 20	Creep and shrinkage
	Coefficients: $\varphi_{t\infty} = 2.55$; $\rho = 0.8$; $\varepsilon_{t\infty} = -24.8 \cdot 10^{-5}$
	Creep-generating permanent load case: 15 The redistribution of internal forces between concrete and prestressing steel are taken into account.

EN 1992-1-1 actions

Standard design group

G - Dead load

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

1 Dead load

CSR1 - Creep, shrinkage, relaxation

Prestressing loss from relaxation of prestressed steel: 4.5 %.

Load cases

20 Creep, shrinkage

G - Additional dead load

Gamma.sup / gamma.inf = 1.35 / 1

Load cases

2 Additional dead load

P - Prestressing

Gamma.sup / gamma.inf = 1 / 1

Load cases internal prestressing

10 Prestressing

QS - Snow and ice load

Gamma.sup / gamma.inf = 1.5 / 0

Combination coefficients for: Superstructures
Snow load - Places in CEN member states with more than 1000 m above sea level
Psi.0 / Psi.1 / Psi.2 = 0.7 / 0.5 / 0.2

Load cases 1. Variante, inclusive

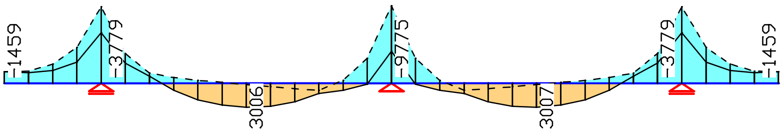
3 Snow load

In this example all possible combinations of load cases are generated and designed. This method is selected in the calculation settings and can be very slow when applied for a large number of load cases.
Below you will find an example of the curve of bending moment M_y for design situations in the ultimate limit states.

1. Permanent and temporary situation - Structural cond.

Construction stage - Ungrouted

G Dead load
P Prestressing

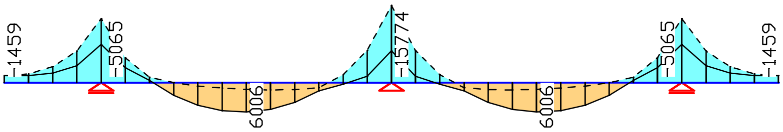


Bending moment M_y [kNm]

2. Permanent and temporary situation - t0

Final state

G Dead load
G Additional dead load
P Prestressing
QS Snow and ice load

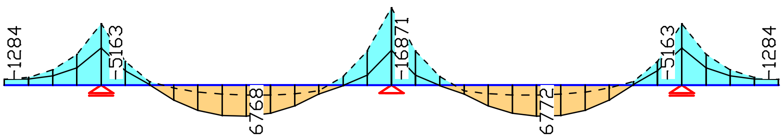


Bending moment M_y [kNm]

3. Permanent and temporary situation - too

Final state

G Dead load
G Additional dead load
P Prestressing
CSR1 Creep, shrinkage, relaxation
QS Snow and ice load



Bending moment M_y [kNm]

Design overview

```
 Se. Expos.   Prestress        Reinforc.  Fatigue      Cr.  De-  Stress
     class    of component     M R B Q T  B Q T P C V  wi.  co.  C B P
   1 XC4      Subsequent bond  x x x x .  . . . . . .   x    .   x x x

(M) Nominal reinforcement to guarantee robustness.
(R) Nominal reinforcement for crack width limitation.
(B) Flexural reinforcement at ultimate limit state, fatigue and stress check.
(Q) (Nominal-)lateral force reinforcement at ultimate limit state and fatigue.
(T) Torsional reinforcement at ultimate limit and fatigue state.
(P) Prestressing steel at fatigue and stress check.
(C) Concrete comp. stress, concrete at fatigue check under long. compression.
(V) Concrete at fatigue check under lateral force.
```

Dispersion of prestressing

```
The variation of prestressing is considered at the following checks:
- Check of decompression and concrete compressive stress
- Nominal reinforcement for crack width limitation
- Check of crack width
All other checks are made using the mean value Pm,t of prestressing.

 Se.  Prestressing of   Const.period   Final state
         component      r.sup  r.inf   r.sup  r.inf
   1  Subsequent bond    1.10   0.90    1.10   0.90
```

Settings for flexural and shear reinforcement

```
M,N     Design mode for bend and longitudinal force:
        (ST) Standard, (SY) Symmetrical, (CM) Compression member.
fyk     Quality of stirrups.
Theta   Angle of concrete truss.
Slabs   Beams are designed like slabs.
Asl     Given reinforcement according to picture 6.3, increase to maximum.
rhow    Factor for minimum reinf. rho.w,min acc. to Chapter 9.3.2(2).
as      Factor for bending reinf. of slabs in secondary dir. per 9.3.1.1(2).
Red.    Reduction factor of prestress for determining the tensile zone for
        distribution of robustness reinforcement for area elements.

                Den-                        Dsn.   Asl [cm²]             Red.
 Se. Concr.     sity     Dsn.   fyk   cot   like   Pic. 6.3     Factor    pre-
                [kg/m³]  M,N   [MPa]  Theta slabs given  max   rhow  as   str.
   1 C45/55-EN        .   ST    500   2.50    .    0.00    .    1.00  .     .
```

Shear sections

```
bw.nom  Nominal width of the prestressed section according to 6.2.3(6).
h.nom   Nominal height of the prestressed section according to 6.2.3(6).
kb, kd  Factor to calculate the inner lever arm z from the eff. width bn resp.
        from the eff. height d.
z1, z2  Height and width of the core section for torsion.
tef     Thickness of the torsion box.
B.      Box section; determination of the bearing capacity acc. to Eq. (6.29).

 Se.   Width  [m]   Eff. width   Height[m]    Eff.height  Torsion. section  [m]
       bw   bw.nom  bn [m] kb     h     h.nom  d [m]  kd    z1     z2    tef  B.
   1  0.500  0.500  0.450 0.90  2.300  2.300  2.250 0.90  2.200  0.400 0.100  .
```

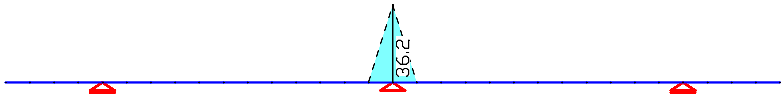
Settings for the check of crack widths

```
ds      Maximal given bar diameter of the reinforcing steel [mm].
max.s   Maximal given bar spacing of the reinforcing steel [mm].
sr,max  Upper limit for the crack spacing from Eq. (7.11) [mm].
Xi1     Bond coefficient of prestressing steel for beam sections.
k       Coefficient for consideration of non-linear distributed tensile stress.
kt      Coefficient for the duration of the load to calculate the crack width.
Fact.   Reduction factor for fctm as per Chapt. 7.3.2 (As) resp. 7.3.4 (wk).
Comb.   Combination for verifying the minimum reinf. (As) and crack width (wk):
        CC, TC, QC = Characteristic, frequent, quasi-continuous combination,
        CT, TT, TB = Centrical tension, tension on top side, tension on bottom,
        CL = Action combination according to exposure class.
Method  Check method for minimum reinf. (kc) and crack width (wk):
kc      Determination of coefficient kc for webs/chords per Eq. (7.2/7.3).
        auto = Eq. (7.2) for rectangular sections, Eq. (7.3) for others.
wk      Calc.   = Direct calculation of crack width as per Chapter 7.3.4,
        Bar sp. = Limiting the bar spacing as per Table 7.3N,
        Cal.(m) = Direct calculation for mean steel strain within Ac,eff,
        Spc.(m) = Lim. the bar spacing for mean steel strain within Ac,eff.
RS      Ring-shaped determination of Ac,eff according to Wiese et al.,
        Beton- und Stahlbetonbau 2004, Issue 4, p 253 ff.

Se.   wmax ds  max  sr   Coefficient   Fact.fctm  Comb.   Method       RS
        [mm]    s  max   Xi1  k   kt   As   wk    As wk  kc    wk
   1  0.20 20    .   .  0.38 0.65 0.4  1.00 1.00  CL CL  auto  calc.    .
```

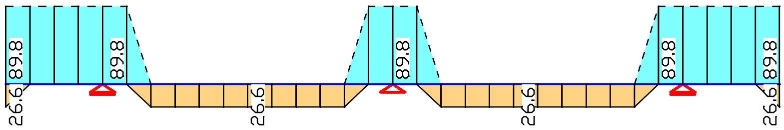
Settings for the check of concrete and steel stresses

```
fck(t)      Compressive strength of concrete at the time t of prestressing.
Sigma.c(t)  Concrete compressive stress at the time t of prestressing.
Sigma.c     Concrete compressive stress in the serviceability limit state.
Sigma.s     Reinforcing steel stress in the serviceability limit state.
(CC),(QC)   Characteristic, quasi-continuous combination.
(TC),(CL)   Frequent combination, combination according to exposure class.

 Se.    fck(t)  per.sigma.c(t)      per.sigma.c    per.sigma.s   Decompression
       [MN/m²]     (CC, QC)          (CC)      (QC)      (CC)      Comb.    Stress
   1     45.00   0.45 fck(t)    0.60 fck       .       0.80 fyk      .         .
```

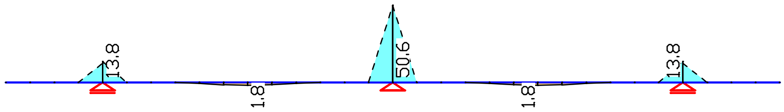
The following illustration shows the curve of the required bending and shear reinforcement.



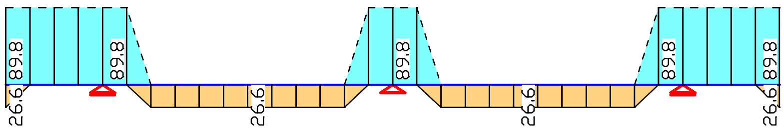
Longitudinal reinforcement A_s from the design in the ultimate limit states [cm²] (upper reinforcement with dashed lines).



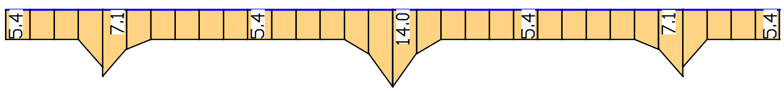
Minimum reinforcement A_s for ensuring robustness (ductility) [cm²] (upper reinforcement with dashed lines).



Reinforcement A_s for limiting the crack width [cm²] (upper reinforcement with dashed lines).



Enclosing reinforcement A_s from the checks [cm²] (upper reinforcement with dashed lines).



(Minimum) lateral force reinforcement $A_{sb,z}$ in the ultimate limit states [cm²/m].

The following pages contain excerpts from the detailed check log for beam 16 at location 2 (middle column).

Design of longitudinal reinforcement

```
The calculated requ. reinforcement includes the specified basic reinforcement.

(M) Nominal reinf. for robustness as per EN 1992-2, 6.1 (109) (Charact. C.)
    fctm     Average centric concrete tensile strength [MN/m²]
    zs,t/b   Lever arm of inner strengths top/bottom with zs=0,9*d [m]
    fyk,t/b  Strength of longitudinal reinforcement top/bottom [MN/m²]
    max Sc   Maximum concrete edge stress from Charact. C. [MN/m²]
             without the statically determined part of prestressing
(R) Nominal/requ. reinforcement as per 7.3.2 for crack width limitation
    Increase of reinforcement due to crack width check is marked by "!".
    wmax     Permissible crack width as per specification [mm]
    ds       Maximal given steel diameter [mm]
    k        Coefficient for consideration of non-linear distributed tensile stress
    fct,eff  Concrete strength at date of cracking [MN/m²]
    kc       Coefficient to consider stress distribution in tensile zone
             acc. to Eq. (7.2) resp. Eq. (7.3)
    Ap'      Part of prestr. steel area Xi1*Ap which was used to reduce req.As
    Xi1      Bond coefficient for prestressing steel as per Eq. (7.5)
    max Sx   Maximal concrete edge stress from action combination [MN/m²]
(B) Design of reinforement at ultimate limit state
    In case of dominant bending, compression reinforcement is marked with "*".
    For section areas acc. to 6.1 (5) the conrecte strain is not limited.
    The minimum reinforcement acc. to 9.2.1.1 and 9.3.1.1 is not determined.
    For compressive members the minimum reinf. acc. to 9.5.2 is considered.
    fck      Concrete strength for design of reinforcement [MN/m²]
    N0, M0   Statically determined forces of tendons with bond [kN, kNm]
    fp0.1k   Charact. value of the 0.1% strain limit of the prestr. steel [MN/m²]
    fpk      Charact. value of the tensile strength of the prestr. steel [MN/m²]
```

```
Beam 16, x = 4.00 m (Beam length 4.00 m)
Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond
Steel 1; Design mode: Standard
(M) fctm=3.8; zs,t/b=2.025/2.025; fyk,t/b=500/500
(R) wmax=0.2; ds=20; kc per Eq. (7.3); k=0.65; fct,eff=3.8; Xi1=0.384
    r.sup/inf(Constr.)=1.1/0.9; r.sup/inf(Final)=1.1/0.9
(B) fck=45

Section properties    A [m²]   ys [m]   zs [m]   Iy [m4]   Iz [m4]   Iyz[m4]
          gross  :     2.926    3.950    0.525    1.2560    9.8822    0.0000
          net    :     2.905    3.950    0.527    1.2535    9.8822    0.0000
          ideally:     2.958    3.950    0.521    1.2596    9.8822    0.0000
```

Tendon groups with bond

```
 No. E-Modul  fp0,1k   fpk       y       z     Ap   Duct    Prestress   Inclin.
     [MN/m²] [MN/m²] [MN/m²]    [m]     [m]  [mm²]  d [mm]        [kN]      [°]
   1  195000   1500    1770   3.950   0.185   7200      82     7555.99     0.00
```

1. Characteristic (rare) combination (CC.1): G.1+P, Construction stage ungrouted

```
Relevant concrete internal forces from 1 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
1    : -7555.93  -4040.19      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
1    : L1+L10
```

2. Characteristic (rare) combination (CC.2): G.1+G.2+P+QS, Final state grouted

```
No set of internal forces in this situation was relevant.
```

3. Characteristic (rare) combination (CC.3): G.1+G.2+P+CSR1+QS, Final state grouted

```
Loss of prestress by CSR in tendon groups
 No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]
   1    9.69          -.-          -.-          -.-          -.-

Stat. determ. part (P+CSR): Nx0=-6823.71 kN; My0=2320.06; Mz0=0.00 kNm

Relevant values from 2 sets of internal forces
         Concrete section              Bond section
Set      Nx[kN]   My[kNm]   Mz[kNm]    Nx[kN]   My[kNm]   Mz[kNm]
2    : -6714.14  -9384.61      0.00    109.56 -11704.67      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : L1+L2+0.96*L10+L20+L3
```

1. Frequent combination (TC.1): G.1+P, Construction stage ungrouted

```
No set of internal forces in this situation was relevant.
```

2. Frequent combination (TC.2): G.1+G.2+P+QS, Final state grouted

```
No set of internal forces in this situation was relevant.
```

3. Frequent combination (TC.3): G.1+G.2+P+CSR1+QS, Final state grouted

```
Relevant concrete internal forces from 4 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
2    : -6042.73  -9624.61      0.00 r.inf

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : L1+L2+0.96*L10+L20+0.50*L3
```

1. Permanent and temporary comb. (PC.1): G.1+P, Construction stage ungrouted

```
Relevant concrete internal forces from 2 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
2    : -7555.93  -4040.19      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : L1+L10
```

2. Permanent and temporary comb. (PC.2): G.1+G.2+P+QS, Final state grouted

```
No set of internal forces in this situation was relevant.
```

3. Permanent and temporary comb. (PC.3): G.1+G.2+P+CSR1+QS, Final state grouted

```
Loss of prestress by CSR in tendon groups
 No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]
   1    9.69          -.-          -.-          -.-          -.-

Stat. determ. part (P+CSR): Nx0=-6823.71 kN; My0=2320.06; Mz0=0.00 kNm

Relevant values from 8 sets of internal forces
         Concrete section              Bond section
Set      Nx[kN]   My[kNm]   Mz[kNm]    Nx[kN]   My[kNm]   Mz[kNm]
2    : -6714.14 -16871.48      0.00    109.56 -19191.54      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : 1.35*L1+1.35*L2+0.96*L10+L20+1.50*L3
```


Design of longitudinal reinforcement

```
Charact. c.: max Sc = 1.63 < 3.80 MN/m² => no minimum crack reinf. required

Reinforcement Nx        My        Mz   max Sc  kc     Ap'    req.As  Situation
Lay. Type    [kN]     [kNm]      [kNm] [MN/m²]        [cm²]   [cm²]
  1   M    109.56 -11704.67      0.00   4.93   .        .     44.91  CC.3,2
      R  -6042.73  -9624.61      0.00    .     .        .     25.28! TC.3,2
      B  -6714.14 -16871.48      0.00    .     .        .     18.11  PC.3,2
  2   M    109.56 -11704.67      0.00   4.93   .        .     44.91  CC.3,2
      R  -6042.73  -9624.61      0.00    .     .        .     25.28! TC.3,2
      B  -6714.14 -16871.48      0.00    .     .        .     18.11  PC.3,2
  3   M      0.06  -6609.23      0.00    .     .        .      0.00  CC.1,1
      R      0.00      0.00      0.00   0.00   .        .      0.00  -,-
      B  -7555.93  -4040.19      0.00    .     .        .      0.00  PC.1,2
  4   M      0.06  -6609.23      0.00    .     .        .      0.00  CC.1,1
      R      0.00      0.00      0.00   0.00   .        .      0.00  -,-
      B  -7555.93  -4040.19      0.00    .     .        .      0.00  PC.1,2
```

Design of shear reinforcement

```
The percentage of nominal reinforcement acc. to Equ. (9.5N) is considered.

Ac         Section area for calculating the concrete stress from long. force [m²]
bw         Effective width for calculation of shear stresses from Qz and Mx [m]
bw.nom     Nominal value of the width when deducting the duct diameter [m]
bn         Statically effective width for shear design using Qy [m]
kb         Factor to calculate the inner lever arm from bn
h          Effective height for calculation of shear stresses from Qy and Mx [m]
h.nom      Nominal value of the height when deducting the duct diameter [m]
d          Statically effective height for shear design using Qz [m]
kd         Factor to calculate the inner lever arm from d
Angle      Angle cot Theta between the compressive strut and the beam axis
Asl giv.   Chargeable longitudinal reinf. acc. to Fig. 6.3 [cm²]
rhow,min   Minimal percentage of lateral reinforcement acc. to Eq. (9.5N)
Qy, Qz     Lateral forces for design in y- and z-direction [kN]
VRdc       Absorbable lat. force without lat. reinf. per 6.2.2 (1) [kN]
VRdmax     Absorbable lateral force of comp. struts per 6.2.3 (3) [kN]
z          Inner lever arm z=kb*bn resp. z=kd*d [m]
Asb.y,z    Req. stirrup reinforcement from Qy resp. Qz [cm²/m]
Asl        Req. longitudinal reinf. acc. to Fig. 6.3 [cm²] for req.Asb
Delta Ftd  Tensile force in long. reinf. from lateral force as per Eq. (6.18)

Beam 16, x = 4.00 m (Beam length 4.00 m)
Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond
bw/bw.nom/bn/kb=0.5/0.5/0.45/0.9; h/h.nom/d/kd=2.3/2.3/2.25/0.9
Ac=2.926; fyk=500; Asl giv./max=0/0; rhow,min=1*(0.08*fck½/fyk)
```

1. Permanent and temporary comb. (PC.1): G.1+P, Construction stage ungrouted

```
No set of internal forces in this situation was relevant.
```

2. Permanent and temporary comb. (PC.2): G.1+G.2+P+QS, Final state grouted

```
Relevant concrete internal forces from 8 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]   Mx[kNm]    Qy[kN]    Qz[kN]
2    : -7555.93 -15774.10      0.00      0.00      0.00  -3029.62

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : 1.35*L1+1.35*L2+L10+1.50*L3
```

3. Permanent and temporary comb. (PC.3): G.1+G.2+P+CSR1+QS, Final state grouted

```
Relevant concrete internal forces from 8 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]   Mx[kNm]    Qy[kN]    Qz[kN]
2    : -6714.14 -16871.48      0.00      0.00      0.00  -3073.60

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : 1.35*L1+1.35*L2+0.96*L10+L20+1.50*L3
```

Check of the shear reinforcement and the compressive struts

```
Action        z           Q/     Asb.y   Asb.z   Asb.T   Asl.T   Asl
max          [m]    Angle VRdc  [cm²/m] [cm²/m] [cm²/m]  [cm²]  [cm²]  Situation
Qy       :  0.41    2.50  0.00      .       .       .       .      .   PC.2,2
Qz       :  2.02    2.50  3.95      .     13.96     .       .     0.00  PC.3,2

Action        z           Qy/     Qz/     Mx/     Q/VRd+    Delta Ftd
max          [m]    Angle VRdmax  VRdmax  TRdmax  Mx/TRd         [kN]  Situation
Qy       :  0.41    2.50  0.00      .       .       .            0.00  PC.2,2
Qz       :  2.02    2.50   .       0.55     .       .         3842.00  PC.3,2
```

Check of crack widths

```
The check is led by direct calculation of the crack width.
The final long. reinforcement as the maximum from robustness, crack and bending
reinf. incl. a possible increase resulting from the fatigue check is decisive.

(CC) Charact. (rare), (TC) Frequent, (QC) Quasi-continuous combination

wmax      Permissible crack width as per specification [mm]
ds        Maximal given steel diameter [mm]
fct,eff   Concrete strength at date of cracking [MN/m²]
Sigma.c   Maximal concrete edge stress in state I [MN/m²]
wk        Calculated value of crack width as per 7.3.4 [mm]
sr,max    Calculated / given maximal crack spacing as per 7.3.4 (3) [mm]
```

```
Ac,eff   Effective region of reinf. [m²] acc. to Fig. 7.1
As,eff   Reinforcing steel within Ac,eff [cm²]
Ap,eff   Prestressing steel with bond within Ac,eff [cm²]
Sigma.s  Reinf. steel stress in state II [MN/m²]
kt       Coefficient for the duration of load as per 7.3.4 (2)
Xi1      Bond coefficient for prestressing steel as per Eq. (7.5)

Beam 16, x = 4.00 m (Beam length 4.00 m)
Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond
wmax=0.2; ds=20; fct,eff=3.8; kt=0.4; Xi1=0.384
r.sup/inf(Constr.)=1.1/0.9; r.sup/inf(Final)=1.1/0.9

Section properties    A [m²]   ys [m]   zs [m]   Iy [m4]   Iz [m4]   Iyz[m4]
          gross  :     2.926    3.950    0.525    1.2560    9.8822    0.0000
          net    :     2.905    3.950    0.527    1.2535    9.8822    0.0000
          ideally:     2.958    3.950    0.521    1.2596    9.8822    0.0000
```

Tendon groups with bond

```
 No. E-Modul  fp0,1k   fpk       y       z     Ap   Duct    Prestress   Inclin.
     [MN/m²] [MN/m²] [MN/m²]    [m]     [m]  [mm²]  d [mm]        [kN]      [°]
   1  195000   1500    1770   3.950   0.185   7200      82     7555.99     0.00
```

1. Frequent combination (TC.1): G.1+P, Construction stage ungrouted

```
No set of internal forces in this situation was relevant.
```

2. Frequent combination (TC.2): G.1+G.2+P+QS, Final state grouted

```
No set of internal forces in this situation was relevant.
```

3. Frequent combination (TC.3): G.1+G.2+P+CSR1+QS, Final state grouted

```
Loss of prestress by CSR in tendon groups
 No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]
   1    9.69          -.-           -.-           -.-           -.-

Stat.determ.part (P+CSR)*r.inf: Nx0=-6141.34 kN; My0=2088.05; Mz0=0.00 kNm

Relevant values from 4 sets of internal forces
         Concrete section              Bond section
Set      Nx[kN]   My[kNm]   Mz[kNm]    Nx[kN]   My[kNm]   Mz[kNm]
2    : -6042.73  -9624.61      0.00     98.61 -11712.67      0.00 r.inf

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : L1+L2+0.96*L10+L20+0.50*L3
```

Check of crack width for reinf. layer 1 (top)

```
Nx        : -6042.73 kN      As,eff  :  89.82  cm²
My        : -9624.61 kNm     Ap,eff  :   0.00  cm²
Mz        :     0.00 kNm     Ac,eff  :   0.988 m²
Sigma.c   :     1.96 MN/m²   Sigma.s :  68.27  MN/m²
Situation : TC.3,2           sr,max  : 509.80  mm
                             wk      :   0.10  wmax 0.20 mm
```

Check of concrete compressive stress

```
For the check, a cracked concrete section (II) is assumed if the tensile stress
from the decisive c. exceeds the value of fctm. Otherwise, a non-cracked section
(I) is used. If the strain is not absorbable on cracked section, (I*) is marked.

fck           Characteristic compressive concrete strength [MN/m²]
fck(t)        Average compressive strength of concrete at time t of the beginning
              of prestressing (Situation G+P) as per 5.10.2.2 (5) [MN/m²]
Sigma.x,min   Total maximal longitudinal compressive stress [MN/m²]
Sigma.x,per   = 0,60*fck for Charact. C. (CC) as per 7.2 (2)
top, bottom   Position of the edge point: above, below of centre

Beam 16, x = 4.00 m (Beam length 4.00 m)
Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond
0.6*fck=27; 0.45*fck(t)=20.25
r.sup/inf(Constr.)=1.1/0.9; r.sup/inf(Final)=1.1/0.9

Section properties    A [m²]   ys [m]   zs [m]   Iy [m4]   Iz [m4]   Iyz[m4]
          gross  :     2.926    3.950    0.525    1.2560    9.8822    0.0000
          net    :     2.905    3.950    0.527    1.2535    9.8822    0.0000
          ideally:     2.958    3.950    0.521    1.2596    9.8822    0.0000
```

Tendon groups with bond

```
 No. E-Modul  fp0,1k   fpk       y       z     Ap   Duct    Prestress   Inclin.
     [MN/m²] [MN/m²] [MN/m²]    [m]     [m]  [mm²]  d [mm]        [kN]      [°]
   1  195000   1500    1770   3.950   0.185   7200      82     7555.99     0.00
```

1. Characteristic (rare) combination (CC.1): G.1+P, Construction stage ungrouted

```
Relevant concrete internal forces from 2 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
1    : -8311.52  -2805.65      0.00 r.sup

Load case combinations for the relevant sets of internal forces
Set    Combination
1    : L1+L10
```

2. Characteristic (rare) combination (CC.2): G.1+G.2+P+QS, Final state grouted

```
No set of internal forces in this situation was relevant.
```

3. Characteristic (rare) combination (CC.3): G.1+G.2+P+CSR1+QS, Final state grouted

```
Loss of prestress by CSR in tendon groups
 No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]
   1    9.69          -.-           -.-           -.-           -.-

Stat.determ.part (P+CSR)*r.inf: Nx0=-6141.34 kN; My0=2088.05; Mz0=0.00 kNm

Relevant values from 4 sets of internal forces
         Concrete section             Bond section
Set      Nx[kN]   My[kNm]   Mz[kNm]    Nx[kN]   My[kNm]   Mz[kNm]
2    : -6042.73 -10509.41      0.00     98.61 -12597.47      0.00 r.inf

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : L1+L2+0.96*L10+L20+L3
```

Check of compressive stress in concrete for the Characteristic (rare) combination

```
Side   Se.-  min Sigma.x  per. Sigma.x  Period   Situation
       Pnt.      [MN/m²]       [MN/m²]
top       2  (I)   -1.68        -20.25  Constr.  CC.1,1
bottom    9  (I)  -16.88        -27.00  Final    CC.3,2
```

Check of steel stress

```
For the check, a cracked concrete section is assumed.
For tendon groups without bond and/or for situations before grouting,
the prestressing steel stress is checked acc. to Eq. (5.43).

Type S  Long. reinf. from N and M, layer number, Charact. C. (CC)
Type P  Prestressing steel, Tendon number, Charact. C. (CC)
N0, M0  Statically determined forces of tendons with bond [kN, kNm]
fck     Concrete strength to determine the strain state [MN/m²]
Sigma.s,per = 0.80 * fyk resp. 1.0 * fyk (CK) as per 7.2 (5)
Sigma.p,per = 0.75 * fpk as per 7.2 (5)

Beam 16, x = 4.00 m (Beam length 4.00 m)
Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond
fck=45; Steel 1; 0.8*fyk,t/b=400/400

Section properties    A [m²]   ys [m]   zs [m]   Iy [m4]   Iz [m4]   Iyz[m4]
          gross  :     2.926    3.950    0.525    1.2560    9.8822    0.0000
          net    :     2.905    3.950    0.527    1.2535    9.8822    0.0000
          ideally:     2.958    3.950    0.521    1.2596    9.8822    0.0000
```

Tendon groups with bond

```
 No. E-Modul  fp0,1k   fpk        y        z     Ap   Duct    Prestress   Inclin.
     [MN/m²] [MN/m²] [MN/m²]     [m]      [m]  [mm²]  d [mm]        [kN]       [°]
   1  195000   1500    1770   3.950   0.185   7200      82     7555.99      0.00
```

1. Characteristic (rare) combination (CC.1): G.1+P, Construction stage ungrouted

```
Relevant concrete internal forces from 2 sets of internal forces
Relevant concrete internal forces from 1 sets of internal forces
Set      Nx[kN]   My[kNm]   Mz[kNm]
1    : -7555.93  -4040.19      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
1    : L1+L10
```

2. Characteristic (rare) combination (CC.2): G.1+G.2+P+QS, Final state grouted

```
No set of internal forces in this situation was relevant.
```

3. Characteristic (rare) combination (CC.3): G.1+G.2+P+CSR1+QS, Final state grouted

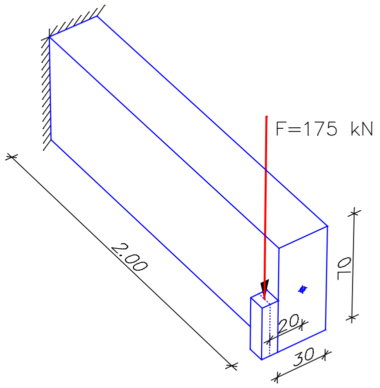
```
Loss of prestress by CSR in tendon groups
 No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]
   1    9.69          -.-           -.-           -.-           -.-

Stat. determ. part (P+CSR): Nx0=-6823.71 kN; My0=2320.06; Mz0=0.00 kNm

Relevant values from 2 sets of internal forces
         Concrete section             Bond section
Set      Nx[kN]   My[kNm]   Mz[kNm]    Nx[kN]   My[kNm]   Mz[kNm]
2    : -6714.14  -9384.61      0.00    109.56 -11704.67      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : L1+L2+0.96*L10+L20+L3
```

Check of steel stress

```
Steel          Nx        My        Mz       As  Sigma.s     per.  Situation
Type No.      [kN]     [kNm]     [kNm]   [cm²]  [MN/m²]   [MN/m²]
 S     1  -6714.14  -9384.61      0.00   44.91    43.29    400.00 CC.3,2
 S     2  -6714.14  -9384.61      0.00   44.91    43.28    400.00 CC.3,2
 S     3  -7555.93  -4040.19      0.00    0.00      .      400.00 CC.1,1
 S     4  -7555.93  -4040.19      0.00    0.00      .      400.00 CC.1,1
 P     1         .         .         .   72.00  1049.44   1275.00 CC.1,--
```

Torsional Beam

The depicted cantilever is subjected to an eccentrically acting load F = 175 kN. The required shear, torsion longitudinal and stirrup reinforcements are listed in the following log.



System drawing

Design according to EN 1992-1-1:2014

Settings for flexural and shear reinforcement

M,N	Design mode for bend and longitudinal force: (ST) Standard, (SY) Symmetrical, (CM) Compression member.
fyk	Quality of stirrups.
Theta	Angle of concrete truss.
Slabs	Beams are designed like slabs.
Asl	Given reinforcement according to picture 6.3, increase to maximum.
rhow	Factor for minimum reinf. rho.w,min acc. to Chapter 9.3.2(2).
as	Factor for bending reinf. of slabs in secondary dir. per 9.3.1.1(2).
Red.	Reduction factor of prestress for determining the tensile zone for distribution of robustness reinforcement for area elements.

Se.	Concr.	Density [kg/m^3]	Dsn. M,N	fyk [MPa]	cot Theta	Dsn. like slabs	Asl [cm^2] Pic. 6.3 given	max	Factor rhow	as	Red. pre-str.
1	C35/45-EN	.	.	500	1.00	.	0.00	.	1.00	.	.

Shear sections

bw.nom	Nominal width of the prestressed section according to 6.2.3(6).
h.nom	Nominal height of the prestressed section according to 6.2.3(6).
kb, kd	Factor to calculate the inner lever arm z from the eff. width bn resp. from the eff. height d.
z1, z2	Height and width of the core section for torsion.
tef	Thickness of the torsion box.
B.	Box section; determination of the bearing capacity acc. to Eq. (6.29).

Se.	Width [m] bw	bw.nom	Eff. width bn [m]	kb	Height[m] h	h.nom	Eff.height d [m]	kd	Torsion section [m] z1	z2	tef	B.
1	0.300	.	0.245	0.90	0.700	.	0.645	0.90	0.590	0.190	0.110	.

Design of shear reinforcement

The percentage of nominal reinforcement acc. to Eq. (9.5N) is considered.

Ac	Section area for calculating the concrete stress from long. force [m^2]
bw	Effective width for calculation of shear stresses from Qz and Mx [m]
bn	Statically effective width for shear design using Qy [m]
kb	Factor to calculate the inner lever arm from bn
h	Effective height for calculation of shear stresses from Qy and Mx [m]
d	Statically effective height for shear design using Qz [m]
kd	Factor to calculate the inner lever arm from d
z1, z2	Height and width of the core section Ak for torsion [m]
tef	Wall thickness of the torsion section [m]
Angle	Angle cot Theta between the compressive strut and the beam axis
Asl giv.	Chargeable longitudinal reinf. acc. to Fig. 6.3 [cm^2]
rhow,min	Minimal percentage of lateral reinforcement acc. to Eq. (9.5N)
Qy, Qz	Lateral forces for design in y- and z-direction [kN]
VRdc	Absorbable lat. force without lat. reinf. per 6.2.2 (1) [kN]
VRdmax	Absorbable lateral force of comp. struts per 6.2.3 (3) [kN]
z	Inner lever arm z=kb*bn resp. z=kd*d [m]
Asb.y,z	Req. stirrup reinforcement from Qy resp. Qz [cm^2/m]
Asl	Req. longitudinal reinf. acc. to Fig. 6.3 [cm^2] for req.Asb
Delta Ftd	Tensile force in long. reinf. from lateral force as per Eq. (6.18)
Mx	Torsional moment for design [kNm]
TRdmax	Maximum absorbable torsional moment as per 6.3.2 (4) [kNm]
Asb.T	Req. stirrup reinforcement from torsion [cm^2/m]
Asl.T	Req. longitudinal reinforcement from torsion [cm^2]
fctd	Design value of the tensile strength for TRd,c in Equ. (6.31) [MN/m^2]

Location 1

Beam 1, x = 0.00 m (Beam length 2.00 m)
Cross-section 1: Polygon - C35/45-EN
Block section z1/z2=0.59/0.19; tef=0.11; fctd=1.49333

1. Permanent and temporary comb. (PC.1): G, Final state

Concrete internal forces

	Nx [kN]	My [kNm]	Mz [kNm]	Mx [kNm]	Qy [kN]	Qz [kN]
Nx- :	0.00	-350.00	0.00	35.00	0.00	175.00
Nx+ :	0.00	-350.00	0.00	35.00	0.00	175.00
My- :	0.00	-472.50	0.00	47.25	0.00	236.25
My+ :	0.00	-350.00	0.00	35.00	0.00	175.00
Mz- :	0.00	-350.00	0.00	35.00	0.00	175.00
Mz+ :	0.00	-350.00	0.00	35.00	0.00	175.00
Mx- :	0.00	-350.00	0.00	35.00	0.00	175.00
Mx+ :	0.00	-472.50	0.00	47.25	0.00	236.25
Qy- :	0.00	-350.00	0.00	35.00	0.00	175.00
Qy+ :	0.00	-350.00	0.00	35.00	0.00	175.00
Qz- :	0.00	-350.00	0.00	35.00	0.00	175.00
Qz+ :	0.00	-472.50	0.00	47.25	0.00	236.25

Check of the shear reinforcement and the compressive struts

Action max, cor.	z [m]	Angle	Q/ VRdc	Asb.y [cm²/m]	Asb.z [cm²/m]	Asb.T [cm²/m]	Asl.T [cm²]	Asl [cm²]	Situation
Qy, Mx :	0.22	1.00	0.00	.	.	.	.	.	-,-
Mx, Qy :	0.22	1.00	0.00	0.00	.	4.85	7.56	0.00	PC.1,Qz+
Qz, Mx :	0.58	1.00	3.04	.	9.36	4.85	7.56	0.00	PC.1,Qz+
Mx, Qz :	0.58	1.00	3.04	.	9.36	4.85	7.56	0.00	PC.1,Qz+

Action max	z [m]	Angle	Qy/ VRdmax	Qz/ VRdmax	Mx/ TRdmax	Q/VRd+ Mx/TRd	Delta Ftd [kN]	Situation
Qy :	0.22	1.00	0.00	.	.	.	0.00	-,-
Qz :	0.58	1.00	.	0.23	.	.	118.13	PC.1,Qz+
Mx :	.	1.00	.	.	0.32	.	.	PC.1,Qz+
Qy "+" Mx:	0.22	1.00	0.00	.	0.32	0.32	.	PC.1,Qz+
Qz "+" Mx:	0.58	1.00	.	0.23	0.32	0.54	.	PC.1,Qz+

Single Design Reinforced Concrete

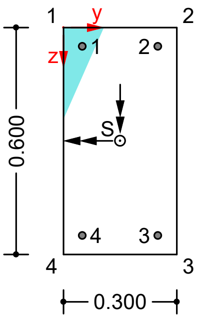
A single rectangular section is designed under bending and normal force.

Pos. 1 - Reinforced concrete design per EN 1992-1-1: 2014

Section 1

Sc. = 1 : 20

Pressure



Action	N = 10.00 kN; My = 67.50; Mz = 27.00 kNm
Resistance	N = 10.00 kN; My = 67.50; Mz = 27.00 kNm
Force system	ys / zs = 0.150 / 0.300 m
Strength	C25/30-EN; gamma.c = 1.50; gamma.s = 1.15
Design mode	Standard
Reinforcement	3.51 cm²; 0.19 %; Concrete area = 1800.00 cm²
Remark	**The concrete compression cannot be checked according to Chapter 6.1 (5). The minimum reinforcement acc. to Chapter 9.2.1.1 (1) is not included.**

Concrete section

Point	y [m]	z [m]	eps[‰]	sigma[MPa]
1	0.000	0.000	-3.50	-16.67
	0.107	0.000	0.00	0.00
2	0.300	0.000	6.31	0.00
3	0.300	0.600	14.98	0.00
4	0.000	0.600	5.18	0.00
	0.000	0.242	0.00	0.00

Inner Forces

	y [m]	z [m]	F [kN]
Compr.	0.030	0.069	-145.43
Tension	0.212	0.518	155.43
Lev. arm	0.181	0.449	

Reinforcement

Point	y [m]	z [m]	d1 [m]	Es,fyk	[MPa]	Zv0 [kN]	eps[‰]	sigma[MPa]	As [cm²]
1	0.050	0.050	0.050	200000	500	0.0	-1.14	-228.48	0.00
2	0.250	0.050	0.050	200000	500	0.0	5.40	437.85	0.23
3	0.250	0.550	0.050	200000	500	0.0	12.63	444.74	2.60
4	0.050	0.550	0.050	200000	500	0.0	6.09	438.51	0.68

Single Design Prestressed Concrete

In this example the results of the prestressed concrete design according to EN 1992-1-1 of the example *Prestressed roof construction* shall be reproduced using the single design according to EN 1992-1-1.

The values relevant for the design can be taken from the detailed listing for beam 16 at location 2 (middle column) of the example.

Location 2

```
Beam 16, x = 4.00 m (Beam length 4.00 m)
Cross-section 1: Polygon - C45/55-EN, 1 tendon group with bond
Steel 1; Design mode: Standard
(B) fck=45
```

Tendon groups with bond

```
 No. E-Modul  fp0,1k   fpk       y       z     Ap   Duct    Prestress   Inclin.
     [MN/m²] [MN/m²] [MN/m²]    [m]     [m]  [mm²]  d [mm]        [kN]       [°]
   1  195000   1500    1770   3.950   0.185   7200      82     7555.99     0.00
```

3. Permanent and temporary comb. (PC.3): G.1+G.2+P+CSR1+QS, Final state grouted

```
Loss of prestress by CSR in tendon groups
 No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]   No.  CSR[%]
   1    9.69          -.-           -.-           -.-           -.-

Stat. determ. part (P+CSR): Nx0=-6823.71 kN; My0=2320.06; Mz0=0.00 kNm

Relevant values from 8 sets of internal forces
         Concrete section              Bond section
Set      Nx[kN]   My[kNm]   Mz[kNm]    Nx[kN]   My[kNm]   Mz[kNm]
2    : -6714.14 -16871.48      0.00    109.56 -19191.54      0.00

Load case combinations for the relevant sets of internal forces
Set    Combination
2    : 1.35*L1+1.35*L2+0.96*L10+L20+1.50*L3
```

Design of longitudinal reinforcement

```
Reinforcement Nx        My        Mz   max Sc  kc     Ap'   req.As  Situation
Lay. Type    [kN]     [kNm]     [kNm] [MN/m²]       [cm²]   [cm²]
  1   B  -6714.14 -16871.48      0.00     .     .       .    18.11  PC.3,2
  2   B  -6714.14 -16871.48      0.00     .     .       .    18.11  PC.3,2
  3   B  -7555.93  -4040.19      0.00     .     .       .     0.00  PC.1,2
  4   B  -7555.93  -4040.19      0.00     .     .       .     0.00  PC.1,2
```

Calculation procedure for the check program:

1. The statically determined part of prestressing with creep and shrinkage ((P + CSR) · cos α · centroid distance) is subtracted from the concrete internal forces.
2. From this the bond internal forces result (statically undetermined part of P + CSR with the internal forces from outer loads).
3. The design is carried out with the bond internal forces. Thereby the prestressing steel together with the loss of prestressing from CSR is taken into account on the resistance side.

Single Design according to EN 1992-1-1

1. For the single design a new section with an additional steel layer at the position of the tendon is necessary. For this purpose section 1 is initially copied to get section 2.
2. Subsequently the new steel layer is added to section 2. The values E, $f_{yk} = f_{P0,1k}$, y, z and $A_s = A_p$ for the check location can be found in the listing. For the prestressing force Z_{v0} the absolute value of the statically determined part N_{x0} from the listing is entered.

Reinforcement of beam elements 2

	E-Modulus [MN/m²]	fyk [MN/m²]	y [m]	z [m]	As [cm²]	Zv0 [kN]
1	210000	500	0.050	0.050	0.000	0.00
2	210000	500	7.850	0.050	0.000	0.00
3	210000	500	4.150	2.250	0.000	0.00
4	210000	500	3.750	2.250	0.000	0.00
5	195000	1500	3.950	0.185	72.000	6823.71

The e-modulus is used for prestressed steel layers only.
y, z coordinates of reinforcement
As Base reinforcement
Zv0 Prestressing force of a prestressed steel layer

3. The bond internal forces with the **statically undetermined** part of P + CSR are necessary for the single design. They can also be taken from the listing.

Single design per EN 1992-1-1 [EBEM_EN1992]

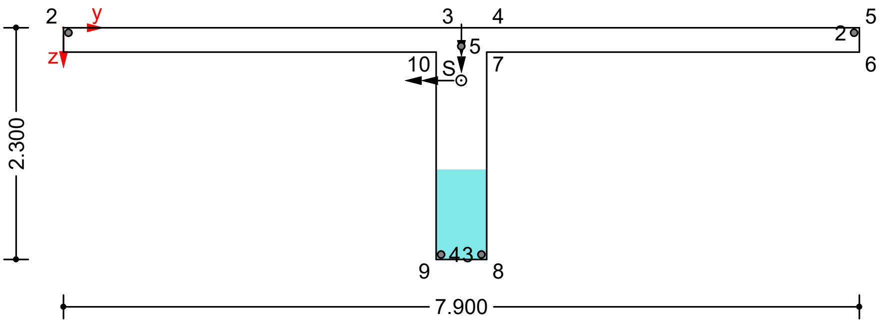
	Section	Combi-nation	Nsd [kN]	Mysd [kNm]	Mzsd [kNm]	Mode
1	2	0	109.56	-19191.54	0.00	Standard
2						

Pos. 1 - Reinforced concrete design per EN 1992-1-1: 2014

Section 2

Sc. = 1 : 75

Pressure



Action	N = 109.56 kN; My = -19191.54; Mz = 0.00 kNm
Resistance	N = 109.56 kN; My = -19191.54; Mz = 0.00 kNm
Force system	ys / zs = 3.950 / 0.525 m
Strength	C45/55-EN; gamma.c = 1.50; gamma.s = 1.15; gamma.p = 1.15
Design mode	Standard
Reinforcement	108.24 cm²; 0.37 %; Concrete area = 29260.00 cm²
Prestress P0	N0 = -6823.71 kN; My0 = 2318.83; Mz0 = -0.00 kNm
Resistance + P0	N+N0 = -6714.15 kN; My+My0 = -16872.71; Mz+Mz0 = -0.00 kNm
Remark	**The concrete compression cannot be checked according to Chapter 6.1 (5).** **The minimum reinforcement acc. to Chapter 9.2.1.1 (1) is not included.**

Concrete section

Point	y [m]	z [m]	eps[‰]	sigma[MPa]
1	-0.000	0.240	4.56	0.00
2	0.000	0.000	5.49	0.00
3	3.700	0.000	5.49	0.00
4	4.200	0.000	5.49	0.00
5	7.900	0.000	5.49	0.00
6	7.900	0.240	4.56	0.00
7	4.200	0.240	4.56	0.00
	4.200	1.405	0.00	0.00
8	4.200	2.300	-3.50	-30.00
9	3.700	2.300	-3.50	-30.00
	3.700	1.405	0.00	0.00
10	3.700	0.240	4.56	0.00

Inner Forces

	y [m]	z [m]	F [kN]
Compr.	3.950	1.928	-10868.30
Tension	3.950	0.165	10977.86
Lev. arm	0.000	1.762	

Reinforcement

Point	y [m]	z [m]	d1 [m]	Es,fyk	[MPa]	Zv0 [kN]	eps[‰]	sigma[MPa]	As [cm²]
1	0.050	0.050	0.050	200000	500	0.0	5.30	437.76	18.12
2	7.850	0.050	0.050	200000	500	0.0	5.30	437.76	18.12
3	4.150	2.250	0.050	200000	500	0.0	-3.30	-435.86	0.00
4	3.750	2.250	0.050	200000	500	0.0	-3.30	-435.86	0.00
5	3.950	0.185	0.185	195000	1500	6823.7	4.77	1304.35	72.00

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